Appendix F Long Term Wastewater Management Strategy, Crandall, 2019a

Greater Shediac Sewerage Commission

Shediac East Long-Term Wastewater Management Strategy
Crandall Project No. 17250-1

TECHNICAL REPORT

Submitted by:

Crandall Engineering Ltd.
1077 St. George Blvd., Suite 400, Moncton, NB E1E 4C9

May 15, 2019

Legal

This report was prepared by Crandall Engineering Ltd. (Crandall) for the Greater Shediac Sewerage Commission (GSSC - Client). This document may not be copied, reproduced, or distributed in any way or for any purpose other than those expressly described in the Client agreement.

The material herein reflects Crandall's engineering judgement based on the information available at the time of preparation, and level of detail as described in the scope of services. Any reliance of a third party on any part of this report is the responsibility of that third party. Crandall accepts no responsibility for claims or damages, if any, resulting from decisions made or actions taken based on use this document by a third party.

<table>
<tr><td colspan="2">Project Name:
Long-Term Wastewater Management Strategy</td><td>Project Number:
17250-1</td><td>Client:
GSSC</td></tr>
<tr><td>Submission Name:</td><td>FINAL</td><td colspan="2" rowspan="5">REGISTERED PROFESSIONAL ENGINEER
Province of New Brunswick
M7653
Shawn E. Burke
Signature
May 15, 2019
Date
Province du Nouveau-Brunswick
INGÉNIEUR IMMATRICULÉ</td></tr>
<tr><td>Revision Number:</td><td>1</td></tr>
<tr><td>Submission Date:</td><td>May 15, 2019</td></tr>
<tr><td>Prepared By:</td><td>Shawn Burke, P.Eng.</td></tr>
<tr><td>Reviewed By:</td><td>Pierre Plourde, P.Eng.</td></tr>
</table>

Executive Summary

In 2014 the Greater Shediac Sewerage Commission (GSSC) completed its Environmental Risk Assessment (ERA) in accordance with Canadian Council of Ministers of the Environment (CCME) guidelines on its main wastewater treatment facility in Cap-Brulé, NB. Resulting from this assessment, the following observations were made of the existing facility:

- Effluent quality was meeting its Certificate of Approval to Operate;
- Effluent quality was meeting the CCME requirements for BOD_5 and TSS;
- The current outfall location does not meet the required mixing levels at the end of its dispersion plume. As a result, a new outfall location was identified off-shore.

While the facility is currently meeting the requirements of its Certificate of Approval to Operate (COA), there are many components that are reaching the end of their service life. Therefore, a review of how the facility will meet future treatment requirements was warranted.

On account of the significant investment required at this facility, Crandall Engineering Ltd. was commissioned by the GSSC to complete this Long-Term Wastewater Management Strategy for the Shediac East area (Cap-Brulé WWTF). The purpose of this study is to complete a comprehensive review of the entire Wastewater Treatment Facility (WWTF) and to provide conceptual design and review of options to upgrade the facility to meet long term needs. This study included the following main tasks:

- Detailed review of previously completed reports and studies;
- Establishment of existing flow and loading conditions at the facility;
- Complete a review of the existing infrastructure and remaining life;
- Review current and future treatment requirements and best practices;
- Review potential impacts of Climate Change (Sea Level Rise) on the existing and future facility;
- Estimation of Long-Term (25-year and 50-year) flows and loading;
- Evaluation of treatment technology options;
- Concept design and cost estimation for treatment technology options.

The following executive summary provides an overview of the scope of this study and presents the key findings and recommendations from the full report. More detailed information and analysis can be found in the full report document, which follows this Executive Summary.

1.1 Existing Conditions

1.1.1 Project Area

The existing Cap-Brulé wastewater treatment facility is located to the east of the Town of Shediac on Cap-Brulé Road, off of Route 133. The influent to the treatment plant enters the site at the south-west corner through two separate connections; one connection from the trunk sewer that brings flows from the Town of Shediac, Shediac Cape, Cap Brulé & Pointe-du-Chêne and one connection from the east that collects flows from the east side of the lagoon. Effluent from the WWTF is discharged into a manmade channel into Lac des Boudreau Ouest by gravity.

Figure 1: Project Area

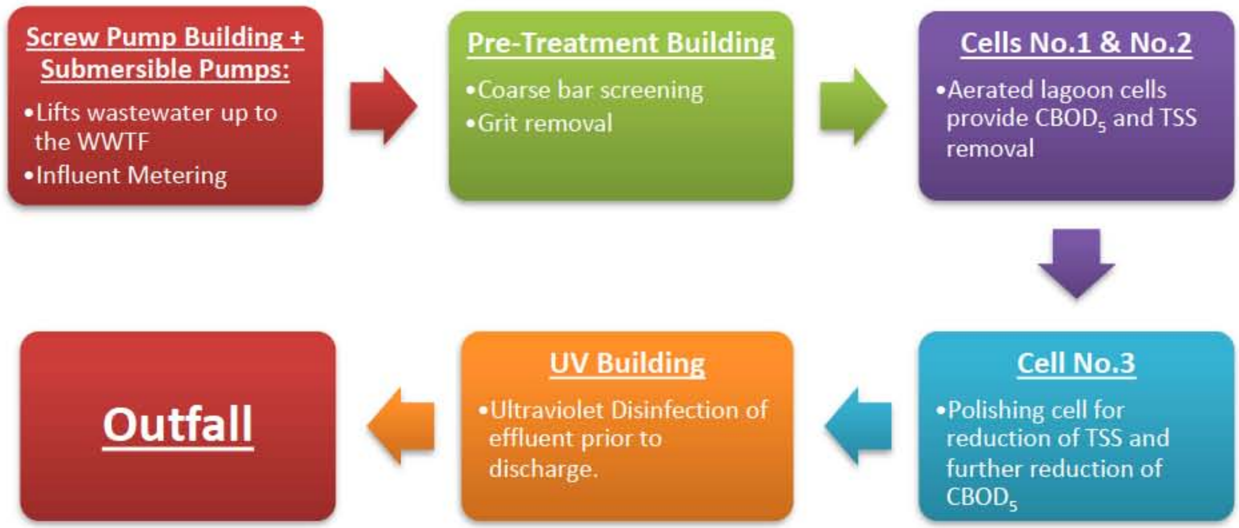
1.1.2 Review of Available Information & Data

In preparation for the study, Crandall gathered and obtained several sources of information, both from previous project records and from the Commission. The following sources of information were reviewed in detail for the purposes of this study:

- GSSC Master Plan (2005)
- WWTF Upgrades (1994) Design Brief
- Shediac West Development Plan (2006)
- Inflow & Infiltration Studies (Various)
- GSSC Wastewater Collection and Treatment (1990)
- Record Drawings (Various)
- CRBE Conceptual Sanitary System Study (2008)
- Water Distribution Master Plan – Shediac (2014)

1.1.3 Existing Infrastructure

The existing Cap-Brulé Wastewater Treatment Facility is an aerated lagoon with pre-treatment and disinfection providing Secondary Treatment levels for the Shediac East area (Town of Shediac, Pointe-du-Chêne, Shediac Cape & Cap Brulé). The following flow chart demonstrates the sequence of treatment through the WWTF:



Please also refer to Drawing 2-1 in the Main Report which provides an overview of the WWTF site.

1.1.4 Site Assessment of Existing Infrastructure

On November 14, 2017 Crandall performed a visual assessment of existing above-ground facilities at the Cap-Brulé WWTF with Mr. Joey Frenette, B.Sc.,PTech – General Manager. This assessment was completed to determine the condition of existing infrastructure at the WWTF and identify current deficiencies or end-of-life components.

As anticipated for this facility, many of the existing assets and major components at the wastewater treatment plant have reached or are nearing the end of their expected useful life. Therefore, regardless of any required upgrade to the WWTF to improve capacity, a major lifecycle renewal is anticipated in the short term (0-5 years).

A detailed list of deficiencies, component lifecycles, and photos can be found in the full report.

1.1.5 Existing Design Criteria and Flows

1.1.5.1 Existing WWTF Design Criteria

The current lagoon was designed for the following design criteria during the WWTF upgrades completed in 1994.

- Design Period: 20 years (1995-2015) – [Based on Loading Projections]
- Average Dry Weather Flow: 6,815 m^3/d (1.8 USmgd)
- Peak Wet Weather Flow: 26,650 m^3/d (7.04 USmgd)

1.1.5.2 Existing Service Population and Flow Rates

To establish the existing treatment plant's capacity to accommodate current and future flows and loading, the existing treatment conditions were reviewed in detail. This involved a review of available flow metering data, influent sampling data, and validation of flow data through theoretical estimates.

(a) Service Population and Historical Growth

The Greater Shediac Sewerage Commission services the Town of Shediac and surrounding areas (including Shediac Cape, Cap Brulé & Pointe-du-Chêne), with a large seasonal variation in population.

Using 2% growth rate on the 2016 Census data to estimate the current (2018) service population results in a figure of 7,475. Please see Section 3.1.1 for discussion on how the 2% growth rate was selected.

(b) Theoretical Flows – Existing Conditions

Theoretical flow estimates were completed for the existing service area to compare theoretical flow estimates with metered flow data, to comment on metered flow patterns.

To estimate existing flows for various development types within the GSSC service boundary, the flow rate allowances (In accordance with the 2006 Atlantic Canada Wastewater Guidelines) were used.

This resulted in the following theoretical flows:

- Total Average Daily Sanitary Flow 6,100 m^3/day
- Total Average Theoretical Daily Flow 7,359 m^3/day
- Peaking Factor 2.65
- Peak Flow 17,872 m^3/day (4.72 MGD).

(c) Metered Flows

When available, metered flow data is often the preferred method for establishing design flows, both average daily and peak, for a wastewater facility. For the Cap-Brulé WWTF, there were several sources of flow metering data available through the commission's SCADA system. Those sources are as follows:

- Pumping Station Data:
- Influent Flow Meter – WWTF
- UV Flow Meter – WWTF

This data was reviewed in detail for the purposes of this study.

From this data, the following analysis was completed:

- Total Average Theoretical Daily Flow 7,997 m^3/day
- Peak Flow: 33,021 m^3/day

1.1.5.3 Selection of Existing Flow Conditions

To establish existing flow conditions for use in analysis of the existing WWTF and future upgrade scenarios, three (3) methods were used, including:

- Method 1: Evaluation of SCADA data at the existing pumping stations and calculating theoretical flows (as described in Section 2.5.2.2) for the area serviced by gravity.
- Method 2: Evaluation of SCADA data at the WWTF (both influent and effluent meters).
- Method 3: Theoretical calculation of flows from entire service area.

The resulting Average daily and Peak flows were calculated as follows:

Table 0-1: Existing Flow - Method Comparison

	Existing Flows	
	Peak Flow (m^3/d)	Average Flow (m^3/d)
Method 1	27,803	7,501
Method 2	33,021	7,997
Method 3	17,872	7,359

While the calculated average daily flow amounts from the three (3) methods are all similar, it was noted that there is a large variation between the calculated peak flows. The noted peak flows for each method range from Method 3 (17,872 m^3/day) for a purely theoretical analysis method, to Method 2 (33,021 m^3/day) which is derived completely from metered data. Method 1, which includes a blend of metered data and theoretical estimates, lies between Method 1 and 2. It is proposed that this is due to the known issue of Inflow and Infiltration (I & I) in the GSSC system.

Following review of the various flow estimations, flows from Method 2 were selected to represent existing conditions. These flows were used to assess existing WWTF capacity as well as for estimating future flow conditions:

Table 0-2: Selected Existing Flows

	FLOW FROM METHOD 2 M^3/D [MGD]	SELECTED FLOW M^3/D [MGD]
Average Daily Flow	7,997 [2.11]	8,500 [2.25]
Peak Flow	33,021 [8.72]	33,100 [8.75]

1.1.5.4 Existing WWTF Loading

Treatment plant loading, in terms of Kg of $CBOD_5$ and TSS per day, is another critical parameter that was established for the purposes of evaluating various concepts and development scenarios.

Loading is a function of effluent concentration multiplied by the daily average flow. Typically, new municipal wastewater treatment plants are designed to a loading of 200/200 kg/day of $CBOD_5$/TSS respectively. However, due to the significant influence that I&I plays on the overall flow patterns at this facility, the loading from existing flow areas had to be accounted for separately.

A review of effluent sampling data from 2016 and 2017 was completed.

While in late summer of 2016, sampling results were in the range of "typical" municipal effluent, average values on an annual basis were highly diluted. Averages over the 2-year sampling period were calculated as:

- Average BOD_5 concentration: 59 mg/l
- Average TSS concentration: 63 mg/l

This results in current average daily loading of 443 / 473 kg of BOD_5/TSS respectively.

1.1.6 Existing Treatment Standards and Effluent Quality

Effluent quality objectives for the existing WWTF have been established through the previously completed Environmental Risk assessment and are stated in the Commission's Certification of Approval to Operate (CAO). The following table summarizes the effluent objectives for this facility, including a summary of the Effluent Discharge Objectives for the future outfall location (with a 1:100 dilution ratio)

Table 0-3: Summary of Effluent Discharge Objectives

Substance	Effluent Discharge Objectives (EDOs)		
	ERA	CAO	New Outfall
TSS	25.0 mg/L	25.0 mg/L	25 mg/L
$CBOD_5$	25.0 mg/L	25.0 mg/L	25 mg/L
Un-ionized Ammonia	1.25 mg/L	1.25 mg/L (maximum)	-
TAN	1.74 mg/L*		29.8 mg/L
TKN			7.3 mg/L
TP			1.7 mg/L
E. *coli*	200 MPN/100ml		200 MPN/100ml

*TAN was selected for on-going monitoring although no treatment is currently provided for this substance and the current Certificate of Approval to Operate does not include an effluent limit. This limit is based on he current outfall location.

The values listed under "New Outfall" were used when evaluating treatment technologies for the proposed upgrades.

1.1.7 Facility Hydraulics

To evaluate the existing facility's hydraulic adequacy, an assessment was completed of the hydraulic gradeline through the facility under various flow scenarios and Tailwater (tide) scenarios using a hydraulic model (SewerCAD).

When the hydraulic model was run under these flow scenarios, there were minor hydraulic issues noted, including an apparent restriction between Cell No.2 and the Polishing Cell.

1.2 Long Term Planning

1.2.1 Flow Conditions

1.2.1.1 Population Growth

To establish the current population (2018) and to estimate future population growth, historical growth rates from Census data were analysed.

As a result of this analysis, a conservative (optimistic) long term growth model of 2% per year was selected for the 50-year scope of this study. The resulting population projection for the Town of Shediac are shown in the following table:

Table 0-4: Projected Population - 50-year Period

YEAR	POPULATION	GROWTH BETWEEN PERIODS	AVG. ANNUAL GROWTH BETWEEN PERIODS
2018 (Current)	7,475	-	-
2043 (25 years)	12,277	4802	2.00%
2068 (50 years)	20,158	7881	2.00%

These population projections were used to establish estimates of future flow and loading rates for the conceptual upgrades to the Cap-Brulé WWTF described herein.

1.2.1.2 Future Growth Areas

A review was completed of the potential growth areas in and adjacent to the Town of Shediac, to determine available land reserve and to comment on whether the projected population growth described in the previous section is possible within these growth areas. For the purposes of this analysis, growth areas were separated between those areas currently within GSSC's service boundary, and other areas. The following areas were reviewed

- Community Rural de Beaubassin Est
- Shediac West
- Infilling Areas

Infilling areas were delineated as those areas which are currently serviced by GSSC or that fall within GSSC's service boundary. These areas were assumed to be collected by GSSC and brought to the Cap-Brulé WWTF.

1.2.1.3 Flow and Loading Projections

Future flow conditions for a 25 and 50-year development period were estimated at the WWTF to select and size various treatment plant upgrade alternatives. This was done by using the selected Existing flow conditions as presented previously and grown according to the population growth projection of 2%.

The following table summarizes the future influent conditions used to complete conceptual design of the various WWTF upgrade options presented later in this report.

Table 0-5: Future Design Flows & Loading Rates - Annual Average

Annual Average Conditions	25-Year		50-Year	
	Average	Peak	Average	Peak
Flow (m^3/day)	12,429	41,855	20,071	55,542
$CBOD_5$ (kg/day)	1,151	-	2,527	-
TSS (kg/day)	1,181	-	2,557	-

1.2.2 Future Regulatory Conditions

To select the target treatment objectives for the future WWTF concept, the study team reviewed treatment requirements with NBDELG, and completed a review of several "best practice" sites. The following target treatment objectives were selected

- $CBOD_5$: 25 mg/L
- TSS: 25 mg/L
- Un-ionized Ammonia: 1.25 mg/L
- TAN: 5.0 mg/L (based on best practice review)
- TP: 1.0 mg/L (based on best practice review)
- E.coli: 200 MPN / 100 mL

1.2.3 Impacts of Climate Change – Sea Level Rise

Whereas the Cap-Brulé WWTF is a coastally located facility and has the potential to be impacted by rising sea levels and/or storm surge, a desktop review was completed of potential impacts to the current and future facilities at this site. Published predictions on future extreme water levels were reviewed in relation to key WWTF components. This is summarized in the following table:

Table 0-6: Summary of Sea Level Rise Impacts to WWTF

Location	Elevation (m)	2030		2100	
		Elev. (m) 1:1 / 1:100	Diff. (m)[1]	Elev. (m) 1:1 / 1:100	Diff. (m)[1]
Outfall	0.71	2.15 / 3.40	-2.69	3.07 / 4.32	-3.61
Metering Chamber	1.96		-1.44		-2.36
UV Channel	3.25		-0.15		-1.07
Top of Berms	Aprox. 4.9		1.5		0.58

1. Negative values denote a surcharged condition.

As shown in the table above, facility components following the UV building are at risk of being impacted by sea-levels during peak events. The potential risks for each component are:

While hydraulic functionality of the WWTF could be impacted by rising sea levels, it appears as though the risk of overtopping the lagoon berms is low. Furthermore, all facilities are well above the 1:100-year return period event in 2100.

Access to the WWTF appears to be unimpacted during an Extreme Sea-level event.

1.2.3.2 Impacts to Future Upgrades

When considering future upgrades, the impacts of sea-level rise should be considered, particularly in the design of the required outfall improvements. It was therefore recommended that the Commission construct a pumped outfall as opposed to a gravity outfall as described in subsequent sections.

1.2.4 WWTF Outfall Options

As a result of the ERA Study, where a new outfall location was recommended to meet the mixing requirements for this facility, a new location discharging directly to the Northumberland Strait was proposed. To construct a new outfall the following three (3) options were presented and analysed:

- Option 1: Gravity Outfall
- Option 2: Pumped Outfall
- Option 3: Status Quo

It was recommended that the Commission pursue a Pumped Outfall as it is preferred due to project cost, cleansing velocities, treatment plant stability (water levels), and the uncertainty of climate change.

1.2.5 Review of Available Treatment Technologies

When evaluating options to service the future needs of GSSC, two (2) main treatment plant types were considered. Under each type, two (2) treatment technologies were reviewed. They were:

- Lagoon Type Treatment Plant
  - Facultative lagoon
  - Aerated Lagoon
- Mechanical Type Treatment Plant
  - Sequential Batch Reactor (SBR)
  - Moving Bed Biofilm Reactor (MBBR)

Each treatment plant type has their benefits, drawbacks and limitations. These two (2) treatment types were reviewed to select the preferred options to be evaluated in more detail through conceptual design.

Furthermore, Additional Treatment Technologies were presented for review during conceptual design. These technologies included pre-treatment, Submerged Attached Growth Reactors (SAGR), MBBR Cells, phosphorus treatment, and UV disinfection.

The following three (3) preferred options were carried forward into conceptual design:

1) Aerated Lagoon Facility
2) MBBR Mechanical-type Facility
3) Hybrid Lagoon/MBBR Facility

1.2.6 Concept Design of Long-Term Treatment Options

Conceptual design was completed for the selected three (3) options, including conceptual design of the required headworks building and outfall, which are required regardless of the option. The options are summarized below:

1.2.6.1 Option 1: Aerated Lagoon

This option involves both re-configuration of existing lagoon cells, and construction of new aerated lagoon cells to meet the long-term flow projections presented previously. To address the requirement for non-acutely-lethal effluent, the addition of a SAGR was proposed. Furthermore, phosphorus treatment would be provided through the addition of alum in one of the lagoon cells, with optional filtration before discharge.

The 25-year concept for this facility has an estimated cost of +/- $30M.

1.2.6.2 Option 2: MBBR Mechanical-type Facility

The selected mechanical treatment process for the proposed upgrades is the Moving Bed Biofilm Reactor (MBBR) technology due to its compact footprint, proven treatment capabilities, ability to treat variable loadings, and less complex operation when compared to other mechanical plant technologies. This option involves construction of a new mechanical treatment plant on the site, and decommissioning of the existing lagoon infrastructure, as it would be redundant.

The 25-year concept for this facility has an estimated cost of +/- $30M.

1.2.6.3 Option 3: Hybrid Lagoon/MBBR Facility

This option employs technology from both a lagoon-type facility and a mechanical-type facility to achieve a concept which allows for re-use of much of the existing lagoon infrastructure, while reducing the footprint requirement substantially when compared to Option 1. Furthermore, because there is no ongoing sludge management required, the operation is significantly less complex than that of Option 2.

The 25-year concept for this facility has an estimated cost of +/- $30M.

1.2.6.4 Comparison of Options

To thoroughly address the options of upgrading the existing lagoon versus constructing a new mechanical plant, consideration needs to be given to various factors, including the anticipated treatment level, land requirements, and operational considerations. The following table summarizes the key considerations of each option:

Table 0-7: Comparison of WWTF Options

CRITERIA	OPTION 1 (LAGOON + SAGR) PHASE 1	OPTION 1 (LAGOON + SAGR) PHASE 2	OPTION 2 (MBBR MECH. PLANT)	OPTION 3 (HYBRID LAGOON/MBBR)
Anticipated Treatment Level				
$CBOD_5$ (mg/L)	15	15	20	15
TSS (mg/L)	20	20	20	15
TAN (mg/L)	1/5*	1/5*	1	2/5*
TP (mg/L)	1/0.3**	1/0.3**	0.5	0.5
E.coli (MPN/100mL)	200	200	200	200
Land Purchase requirements (Ha)	2.6	10.5	0	0
Operational Stability	Very stable		Stable	Stable
Operation & Maintenance Requirements	Normal		More Advanced	Moderately
Operator Training	Simple		More Complex	Advanced
Operational Complexity	Removal (±15 years)		Continuous	Moderately Complex
Sludge Handling	One (Alum)		Several (anti-foam,	Removal (2-3 yrs)
Chemical Use			coagulant, polymer, acid)	One (Coagulant)
Capital Cost	\$30M***		\$30M	\$30M***

* summer / winter

** without filtration system (addition of alum between lagoons only) / with filtration system

*** Cost for Phase 1.

Based on this analysis, it was recommended that the Commission proceed to preliminary design of **Option 3: Hybrid Lagoon/MBBR Facility.** This option was recommended due to the operational flexibility and reduced operation and maintenance requirements. Furthermore, this option re-uses the existing lagoon cells at the WWTF.

1.2.7 Conclusions and Recommendations

The following section summarizes the key conclusions and recommendations presented in the report. Please see the full report text for the full list:

1. The existing facility, while continuing to produce effluent results consistent with the Certificate of Approval to Operate (CAO), is approaching its design capacity. This is consistent with the design life of the upgrades completed at this facility in 1994. Therefore, improvements will likely be required in the short term to continue to meet the CAO objectives.

2. Through a review of the major components at the existing WWTF, it is evident that many of the components installed during the last life-cycle upgrade (1994) are reaching the end of their service life and will require attention in the short term (0-5 years).

3. As one of the results of the recently completed Environmental Risk Assessment (ERA), the outfall is currently not meeting the CCME requirements for mixing levels. As recommended in the subsequent report entitled *Feasibility Study: Cap-Brulé WWTP Outfall (2015)* a new outfall location approximately 350m off-shore is required to achieve the required mixing ratios for the facility. This upgrade should be completed in the short term (0-5 years), whether part of an overall upgrade of the WWTF or independently, to remain in compliance with CCME requirements. If completed independently, it is recommended that the required infrastructure be located according to the concepts presented herein for the overall WWTF upgrade.

4. Options for the new required outfall, as previously presented in Feasibility Study: Cap Brulé WWTP Outfall (2015), were re-visited in light of the overall concept for site upgrades. It is recommended that the Commission consider a pumped outfall as the preferred solution. The reasons for this recommendation include:

   a) Constructability of a pressure pipe option is better than a larger gravity pipe.
   b) Difficulties in accommodating the required diffusers at the end of the outfall due to headloss limitations with a gravity option.

c) Water level in the facility is directly influenced by sea-levels in the gravity option and is therefore sensitive to the impacts of climate change.

d) Concerns with maintenance of a gravity option due to lower velocities through the larger required pipe.

5. Available treatment technologies were reviewed in detail for the required WWTF upgrades. Primarily, a comparison was made between a lagoon-type facility, a mechanical-type facility and a hybrid Lagoon/MBBR facility. Through an evaluation of the comparative costs and benefits of each facility type, it is recommended that the Commission proceed to preliminary design with a Hybrid option. The following additional recommendations are presented:

   a) It is recommended that the project team evaluate the merit of designing any facility components for the 50-year design flow projections presented herein. It is proposed that due to the uncertainty of these projections, and their impact on the overall scale and cost of the required upgrades, that preliminary design proceed for the 25-year design flow projection.

   b) Due to the magnitude of the recommended upgrades, it is proposed that a detailed review of phasing options be completed during preliminary design activities. It is likely that the Commission will be able to partition this project into phases that meet the current needs of the WWTF in the short term while positioning themselves to meet the full 25-year concept in the medium term.

6. Order of magnitude cost estimates were established to assist in comparing each option. These estimates include a contingency (20%), an allowance for engineering (15%) and allowances for environmental and geotechnical studies. The estimated costs are summarized below:

   a) Lagoon Type Facility
      - Phase 1: 25-year Concept: $30M

   b) Mechanical Type Facility
      - Phase 1: 25-year Concept $30M

   c) Lagoon/Mechanical Hybrid Concept
      - Phase 1: 25-year Concept $30M

   d) Preliminary Design: $150 – 200k (scope to be confirmed)

7. It is recommended that the Commission proceed to preliminary design immediately following selection of the preferred conceptual option. Due to the nature of the required upgrades, there are considerable investigation, permitting and design activities that are required prior to commencing construction of the WWTF improvements. Furthermore, completing preliminary design would allow the Commission to be positioned to request funding through the next round of the Building Canada Fund (BCF), which is anticipated to open for applications in the Fall of this year (2018).

TABLE OF CONTENTS

DRAWINGS

FIGURES

TABLES

APPENDICES

1 Introduction

In 2014 the Greater Shediac Sewerage Commission (GSSC) completed its Environmental Risk Assessment (ERA) in accordance with Canadian Council of Ministers of the Environment (CCME) guidelines on its main wastewater treatment facility in Cap-Brulé, NB. The purpose of an ERA is to evaluate the current treatment objectives established by the regulatory authority and to confirm if they are adequately protecting the receiving environment. Resulting from this assessment, the following observations were made of the existing facility:

- Effluent quality was meeting its Certificate of Approval to Operate;
- Effluent quality was meeting the CCME requirements for BOD_5 and TSS;
- The current outfall location does not meet the required mixing levels at the end of its dispersion plume. As a result, a new outfall location was identified off-shore.

Because of these findings, in 2015 the GSSC commissioned a feasibility study for a new outfall that would meet the CCME mixing requirements. As part of the ERA, it was found that an acceptable mixing zone was available in the Northumberland Strait, approximately 350m off shore. Due to the hydraulic losses through this outfall, the soft soils and environmental issues, the estimated cost to complete this upgrade is significant. While the facility is currently meeting the requirements of its Certificate of Approval to Operate (COA), there are many components that are reaching the end of their service life. Therefore, a review of how the facility will meet future treatment requirements was warranted.

On account of the significant investment required at this facility, Crandall Engineering Ltd. was commissioned by the GSSC to complete this Long-Term Wastewater Management Strategy for the Shediac East area (Cap-Brulé WWTF). The purpose of this study is to complete a comprehensive review of the entire Wastewater Treatment Facility (WWTF) and to provide conceptual design and review of options to upgrade the facility to meet long term needs. This study included the following main tasks:

- Detailed review of previously completed reports and studies;
- Establishment of existing flow and loading conditions at the facility;
- Complete a review of the existing infrastructure and remaining life;
- Review current and future treatment requirements and best practices;
- Review potential impacts of Climate Change (Sea Level Rise) on the existing and future facility;

- Estimation of Long-Term (25-year and 50-year) flows and loading;
- Evaluation of treatment technology options;
- Concept design and cost estimation for treatment technology options.

The following report provides a detailed review of the study methodology, assumptions and design considerations made in order to provide the GSSC with recommendations related to the continued operation of a wastewater treatment plant on this site.

Figure 1-1: Cap-Brulé WWTF

2 Existing Conditions

2.1 Project Area

The existing Cap-Brulé wastewater treatment facility is located to the east of the Town of Shediac on Cap-Brulé Road, off of Route 133 (PID 01065655 and 01065663). The facility is bordered by Route 133 to the south, Chemin Cap-Brulé to the west, Lac des Boudreau Ouest to the north and an undeveloped rural residential area to the east. The influent to the treatment plant enters the site at the south-west corner through two separate connections; one connection from the trunk sewer that brings flows from the Town of Shediac, Shediac Cape, Cap Brulé & Pointe-du-Chêne and one connection from the east that collects flows from the east side of the lagoon.

Effluent from the WWTF is discharged into a manmade channel into Lac des Boudreau Ouest by gravity.

Figure 2-1: Project Area

The wastewater treatment facility on this site has evolved and has been upgraded over several projects since the original lagoon was constructed here in 1970. Projects have included:

- 1971-72 (original) Two cell facultative lagoon with surface aerators
- 1994-96 Improvements to the Regional Wastewater Treatment Plant, separated into five (5) contracts which included a new pre-treatment building, dividing one of the original lagoon cells into two aerated cells, and a new blower building.
- 2004 Submersible pumping station (LS 24) constructed next to the screw pump building
- 2009 Wastewater Treatment Plant Upgrade - Disinfection (UV) Unit
- 2012 Screw pump upgrades

2.2 Review of Available Information & Data

In preparation for the study, Crandall gathered and obtained several sources of information, both from previous project records and from the Commission. The following section summarizes the information provided and its relevance to this report.

2.2.1 Master Plan

The recent master plan edition for the GSSC was commissioned in 2013 to update the past GSSC master plan (completed in 2005) with all the new infrastructure improvements both with the sanitary system as well as the storm and water system in partnership with the Town of Shediac. The current master plan is a combination of old sewer mapping information and new GIS coordinated information. This plan is used by the GSSC for locating infrastructure information and to aid in asset management of the present system.

2.2.2 GSSC Wastewater Collection and Treatment Study

In March of 1990, Crandall Engineering Ltd. was requested by the GSSC to investigate the physical condition of the existing wastewater collection and treatment system within the Commission's boundaries. The major objective of this study was to complete a thorough investigation of the Commission's system for future upgrading and system expansion.

This study was reviewed and referenced for information such as the historical WWTF capacity and flows. Furthermore, there is some discussion of summer population increases, inflow an infiltration in the system and loading. This information was used when preparing flow and loading estimates in this study.

2.2.3 WWTF Upgrades (1994) Design Brief

This report built on the findings from the 1990 report and describes the design conditions and criteria for the major WWTF upgrades completed between 1994 and 1996. The facility described in this report represents the current conditions and had a design life of 20 years (2015) based on projected flows and loading rates. However, due to construction efficiencies related to the configuration of the original lagoon, the aerated cells were proposed to be constructed 25% larger than required for estimated design flows.

2.2.4 Record Drawings

Various construction and record drawings were referenced from the various capital projects that have occurred at the WWTF to confirm information on existing conditions. This information was used throughout the study for tasks including assessment of the existing plant capacity and hydraulics.

2.2.5 Shediac West Report

This report, submitted in June of 2006, reviewed the potential for development and future servicing (sanitary sewer collection and treatment) of the area to the west of the Town of Shediac. As part of the NB Dept. of Environment and Local Government's review of a major proposed development in this area, the Department requested that this study be completed. This area is presently mainly farmland but there has been an increasing interest in developing this land in recent years. It was determined that it was not economically viable to connect this future development area into the current GSSC system.

This report was referenced with respect to future population growth projections for the Town of Shediac.

2.2.6 Communauté Rurale de Beaubasin Est - Conceptual Sanitary System Design Study

This 2008 study explored options for wastewater collection and treatment to service the rural community of Beaubasin Est, which is currently on the eastern limit of the GSSC service boundary. This study made recommendations on treatment plant location(s), collection system requirements, phasing and provided an estimate of overall cost. The study recommended the long-term development of two lagoons and did not evaluate the potential for connection with the Cap-Brulé WWTF.

2.2.7 Water Distribution Master Plan

This Master Plan was completed to update the original water system master plan completed in 1999. More specifically, the objective was to review existing and future water demands, perform an analysis of the water distribution system and to identify deficiencies and proposed improvements to the Town's system. As part of this study was to make recommendations on water supply and storage, an analysis was completed of population projections and water usage patterns. This information was reviewed in relation to this study to align on future population projections.

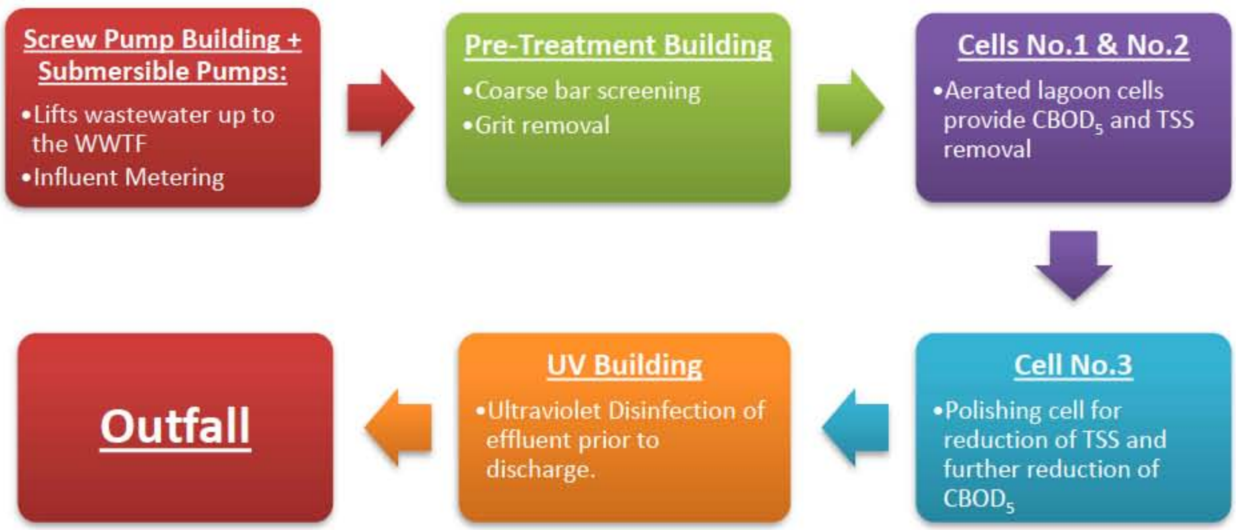
2.2.8 Inflow and Infiltration Studies – Various

Inflow and infiltration have historically been, and continue to be, significant issues affecting the GSSC's collection and treatment systems. Several studies and projects have been completed over the years in an attempt to address this issue.

Please see Section 2.5.4 for additional discussion.

2.3 Existing Infrastructure

The existing Cap-Brulé Wastewater Treatment Facility is an aerated lagoon with pre-treatment and disinfection providing Secondary Treatment levels for the Shediac East area (Town of Shediac, Pointe-du-Chêne, Shediac Cape & Cap Brulé). The following flow chart demonstrates the sequence of treatment through the WWTF:



The following sections describe the various components in greater detail. Please also refer to Drawing 2-1 on the following page which provides an overview of the WWTF site.

2.3.1 Existing Gravity Sewer

The main collector for the WWTF is a 4.5 km long gravity trunk sewer that starts at the Federal public service pension building and outfalls to the GSSC WWTF at the Screw Pump Wet Well (See Drawing 2-2 on the following page showing existing trunk sewer routing).

The trunk sewer was renewed in 2010 as the previous trunk sewer had reached its hydraulic capacity. The present trunk sewer enters the facility as a 900mm diameter reinforced concrete pipe at a 0.05% grade into the screw pump wet-well with sluice-gates to either divert flow to the screw pump wet well or the submersible pump wet well. In high flow events all sluice-gates are kept open to use both sets of pumps in parallel. The gravity trunk sewer invert entering the WWTF is – 1.650m geodetic (Approximate 7m Deep from the surface.

2.3.2 Influent Lift Station

The wastewater flows received at the WWTF through the gravity trunk sewer are currently pumped up through a combination of screw pumps and submersible pumping. After being pumped, wastewater flows by gravity through the remainder of the WWTF. The pumping components are described as follows:

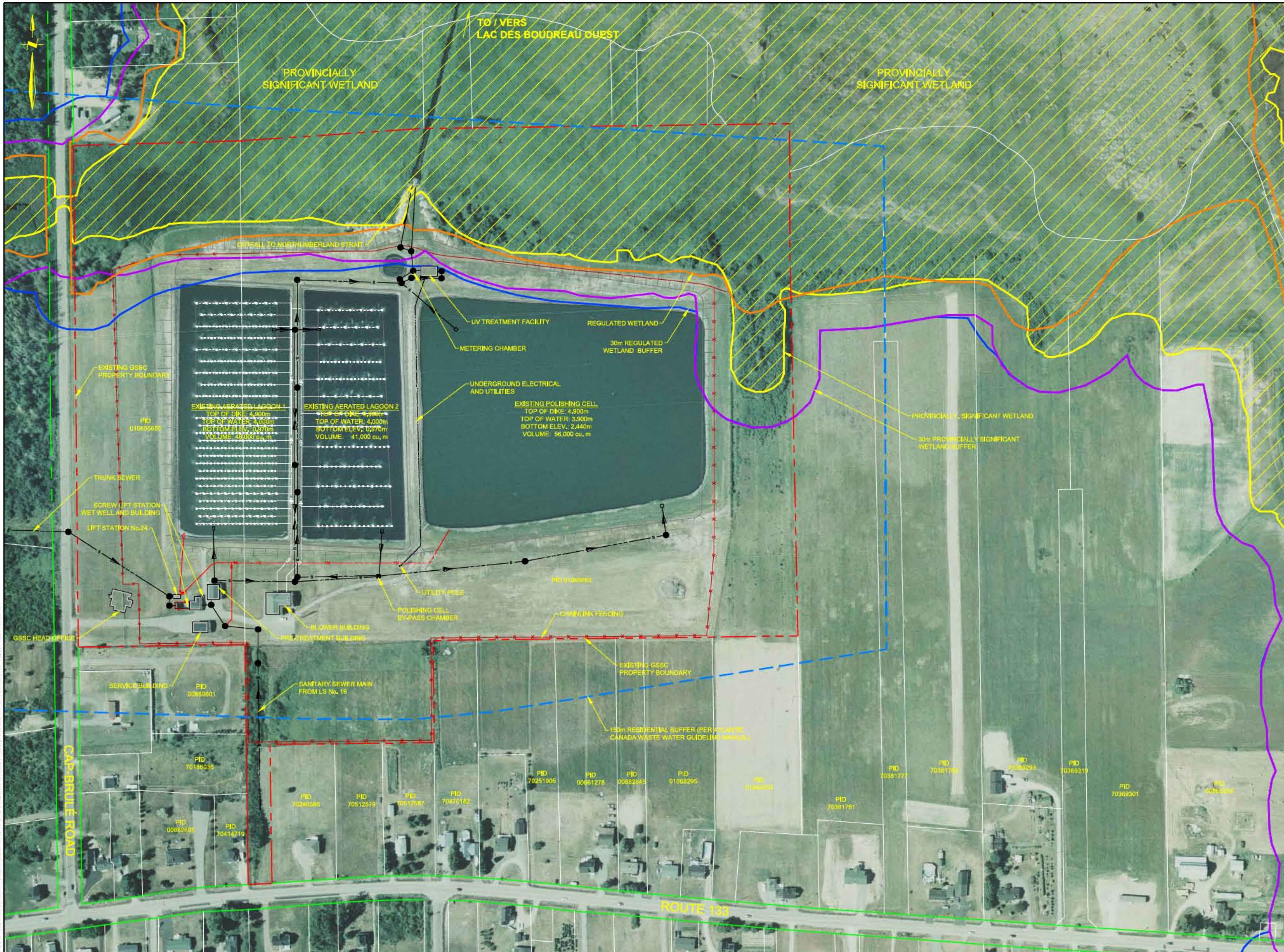
2.3.2.1 Screw Pumps

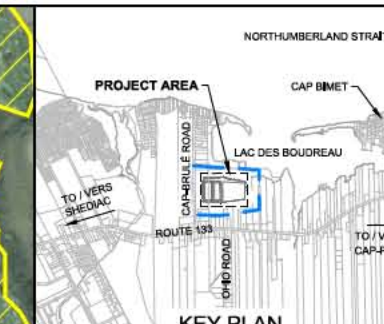
The screw pumps are located as shown on Drawing 2-1. Screw pump bodies are located outdoors with the drive units in an enclosed, heated building.

There is a total of three (3) screw pumps in operation. The original screw pumps were installed in 1971 and recently screw pumps 1 and 2 were replaced with a new screw pump assembly in 2012 that included new screws, upper and lower bearings, metal troughs, profile plates, electric motors and gearboxes.

The Concrete wet well structure surrounding the screws has deteriorated over the last 47 years. Many areas of the concrete walls exhibit superficial deterioration. A structural assessment was completed in 2011 by Valron Engineers Inc. and it was concluded that it was reaching the end of its useful life.

An electric motor and gear at the top turns each screw individually at low speed (40-60 rpm). This turning motion draws the water upwards between the flights until it spills over the trough at the top. These pumps are very effective as they can pump at different rates of flow depending on how high the water is in the wet well, more inlet flow more pumping capacity with no change in screw speed. Each screw pump is powered by a 15kW motor and has a capacity of 95 l/s (total design capacity of 285 l/s). However, it is expected that the capacity of the original screw pump (1 of 3) has been reduced substantially because of deterioration in the channel.





KEY PLAN
N.T.S.

LEGEND

- PRIMARY FLOW PATTERN
- SANITARY MANHOLE
- PROVINCIALLY SIGNIFICANT WETLAND
- PROVINCIALLY SIGNIFICANT WETLAND (30m BUFFER)
- REGULATED WETLAND
- REGULATED WETLAND (30m BUFFER)

NO.	DATE	REVISIONS	BY	APPR.
1.0	NOV 23/18	ISSUED FOR FINAL REPORT	GMG	SEB
0.0	JUL 10/18	ISSUED FOR DRAFT REPORT	DWD	SEB

GREATER SHEDIAC SEWERAGE COMMISSION DES ÉGOUTS SHEDIAC ET BANLIEUES

crandall

PRELIMINARY ONLY
NOT TO BE USED FOR CONSTRUCTION

PROJECT TITLE

SHEDIAC EAST LONG-TERM WASTEWATER MANAGEMENT STRATEGY

SHEDIAC N.B.

DRAWING TITLE

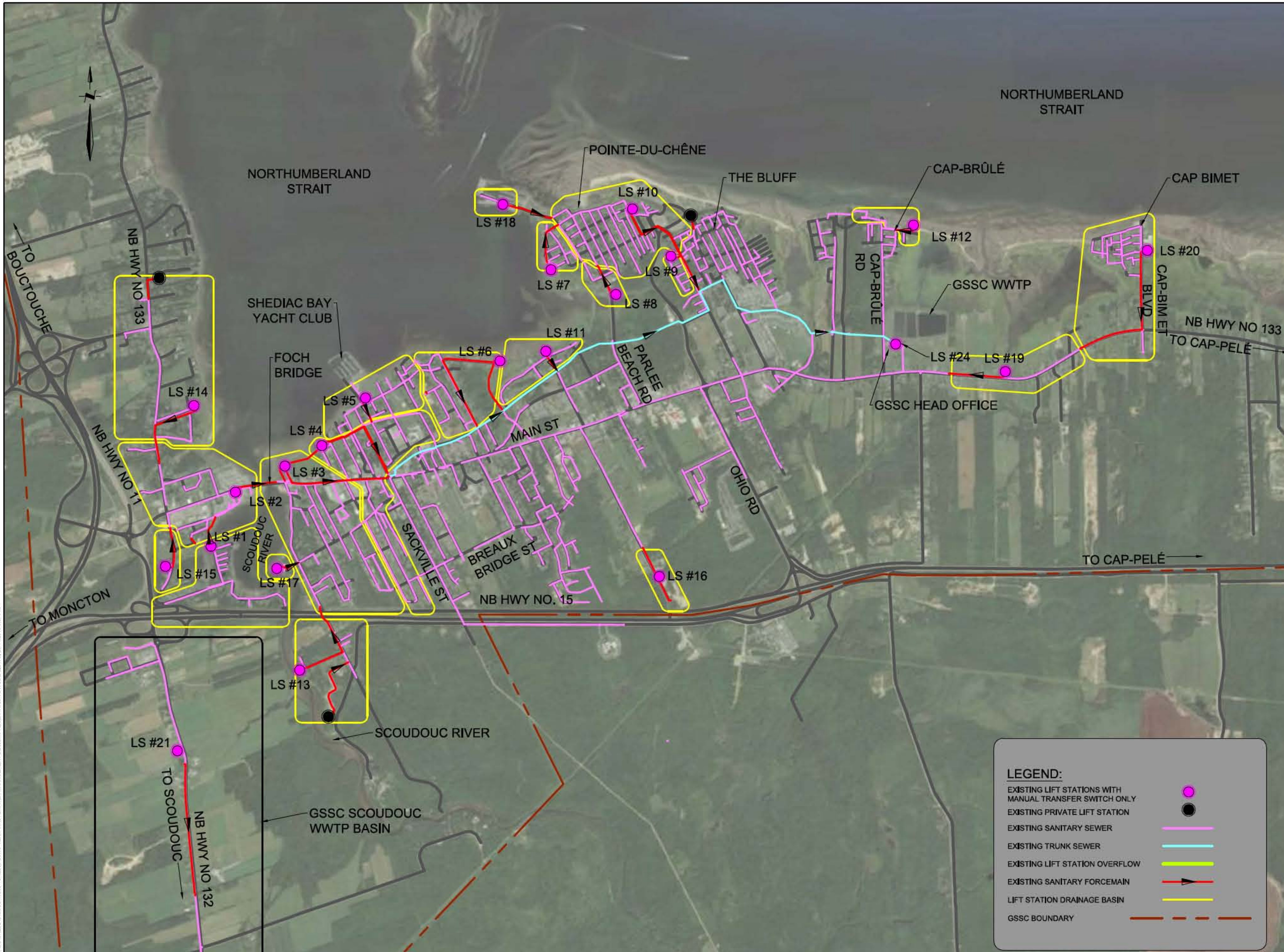
EXISTING WWTF CONDITIONS

Scale	Drawn By	Design By
15m 0 30m SCALE : 1:3000	GMG	SEB
	Checked By PP	Cadd Check BW
	Sheet 1 of 1	

File Name 2-1.DWG

Drawing No. 2-1

O:\CADD\SHEDIAC\GSSC17250 - SHEDIAC LONG-TERM WASTEWATER MANAGEMENT\DESIGN\PRESENTATION DWGS\2-1.DWG, 23/11/2018 3:06 PM



NOTES

NO.	DATE	REVISIONS	BY	APPR.
1.0	NOV 23/18	ISSUED FOR FINAL REPORT	GMG	SEB
0.0	JULY 10/18	ISSUED FOR DRAFT REPORT	DWD	SEB

GREATER SHEDIAC SEWERAGE COMMISSION
DES ÉGOUTS SHEDIAC ET BANLIEUES

crandall

PRELIMINARY ONLY
NOT TO BE USED FOR CONSTRUCTION

PROJECT TITLE

SHEDIAC EAST LONG-TERM WASTEWATER MANAGEMENT STRATEGY

SHEDIAC N.B.

DRAWING TITLE

EXISTING COLLECTION SYSTEM OVERVIEW

Scale	Drawn By	Design By
150m 0 300m SCALE : 1:30 000	JHK	CG
	Checked By: CG	Cadd Check: BW
	Sheet 1 of 1	

File Name: 2-2.DWG

Drawing No. 2-2

At the upper end of the screw pumps, the waste water flows through a Parshall flume with a 900mm opening. This with the ultrasonic level sensor installed in 1993 measure the flow into the WWTF.

2.3.2.2 Submersible Pumps

In Addition to the screw pumps, in 2004/2005 a triplex submersible pumping station was installed next to the existing screw pumps wet well. As mentioned in section 2.3.1., the gravity sewer can be directed to either wet well via sluice gates.

The Submersible Station (Lift Station No. 24) is a triplex system with submersible Flygt Pumps. Each pump is powered with a 30 hp motor and has a capacity of 66 l/s (1050 USgpm).

The submersible station was added to the WWTF to run in parallel with the Screw pumps for two (2) main reasons:

1. When maintenance is required for the screw pumps or vice versa, sluice gates a can be closed, and the wet well requiring maintenance can be dewatered with no disruption of the influent flow.
2. During high flow events where the screw pump reaches capacity the submersible pumps are automatically activated to increase capacity during peak periods. This is controlled via the SCADA system and level sensor installed in the screw pump wet well. The submersible system has three (3) options for where it can discharge the extra flows, either directly to Cell #1, Cell #3 or directed back to the pre-treatment building using a network of valves.

2.3.3 Pre-Treatment Building

The Cap-Brulé WWTF is equipped with pre-treatment equipment to remove inorganic solids that are characteristic of municipal effluent. This equipment protects the remaining WWTF components from these solids, which can accumulate in the lagoon if not adequately removed. The various system components are described in further detail below.

2.3.3.1 Bar Screen

Upon entering the pre-treatment building, wastewater normally first passes through a mechanically raked vertical bar screen (Cont-Flo vertical bar screen) screen in the main channel. The screen was designed for the following operating conditions:

- Maximum Flow: 26 650 m^3/d
- Maximum Water Level: 700 mm
- Channel width: 750 mm
- Screening width: 750 mm
- Channel depth: 1,220 mm
- Bar Spacing: 38.1 mm

The raking system which removes accumulated debris from the screen operates on a timed interval and is initiated if the differential water level setting between the upstream and downstream sides of the screen is exceeded.

2.3.3.2 Vortex Grit Removal & Dewatering

Following the bar screen, flow is directed to a grit chamber system (MECTAN model JMD/3-30) to remove sand, gravel, grit and other non-organic debris prior to wastewater entering the first lagoon cell. The grit removal system installed in the GSSC pre-treatment building includes the following components:

- Grit Well: This 3000mm diameter chamber is designed to provide a quiescent zone to allow settlement of grit.

- **Grit Agitator:** To keep organic material in suspension through the grit removal system, a steel paddle rotates through the chamber at a constant rate of 19 RPM.
- **Grit Extraction:** The grit extraction system consists of the following components:
  - Fluidisation: A water tank and pump system is installed to inject 55 USgpm of water at 90 psi at the bottom of the grit well to "fluidise" the accumulated grit before extraction.
  - Air Scour: During a grit extraction cycle, air is injected into the base of the grit chamber to free the organic matter that may have settled in the grit well.
  - Airlift Pump: This airlift system evacuates accumulated grit from the grit well up to the dewatering screw.
- **Dewatering Screw:** Evacuated grit is brought to a "SAM" grit dewatering screw system where excess liquid is removed from the grit. Dewatered grit is then ready for storage and subsequent disposal.

This system was designed for the following operating conditions

- Maximum Flow: 26 650 m^3/d
- Required Water Level: 750 mm

2.3.3.3 Sluice Gates and By-pass Channels

Channels through the pre-treatment building are configured to allow the operator to bypass either the bar screen or grit removal system or both through operation of sluice gates. This facilitates maintenance of these components.

2.3.4 Lagoon Cells

Originally constructed as two (2) large facultative lagoon cells with surface aerators, the first (west) cell was split into two (2) aerated cells as part of upgrades at the site in 1995. The remaining large cell was left as a shallow polishing cell. Physical information on the current cells is shown in the table below:

Table 2-1: Physical Properties - Existing Lagoon

Parameter	Aerated Cell #1 (West)	Aerated Cell #2 (East)	Polishing Cell
Surface Area	1.88 ha	1.64 ha	3.79 ha
Surface Elevation	4.0 m	4.0 m	3.90 m
Bottom Elevation	0.97 m	0.97 m	2.44 m
Liquid Depth	3.03 m	3.03 m	1.462 m
Liquid Volume	47,677 m^3	40,576 m^3	52,191 m^3
Liner	Membrane	Membrane	Clay

2.3.5 Aeration System

As part of the 1994 upgrades, a coarse bubble aeration system was added to the first two cells. This system was installed to increase the treatment efficiency while maintaining the same plant footprint. Air flow can be adjusted to accommodate the large variations in flow and loading, particularly to address the increase in loading during the tourist season. This is done through monitoring of dissolved oxygen sensors installed in each cell. The increased oxygen supply is required to address the increased loading rates during these periods.

The following aeration components were installed during the upgrade:

2.3.5.1 Blower Building & Centrifugal Blowers

A multipurpose building was constructed to house the new blowers, as well as provide vehicle storage, and lab facilities. The blower system was designed to provide the required air flow for the 20-year design life through two (2) blowers and one (1) standby unit that is included in the operational cycle. Ultimate design includes provision for a fourth blower to be added, whereby three (3) blowers would supply the oxygen demand and the fourth would be a standby. At the time of this study, three (3) blowers are installed. Oxygen supply was selected as follows:

- 20-year design oxygen requirement: 2,237 kg/day
    - Required Air Flow: 2,394 l/s
    - Total blower capacity: 2,992 l/s (125% of required)
    - Target $CBOD_5$ removal: 11,18 kg/day (93.2% removal eff.)
- Ultimate design oxygen requirement: 2,796 kg/day (125% x 2237)
    - Required Air Flow: 2,970 l/s
    - Total blower capacity: 3,712 l/s (125% of required)
    - Potential $CBOD_5$ removal: 1,397 kg/day
- Blower Selection 1,240 l/s @ 125 hp

As discussed in Section 2.5.3 the current peak daily loading at the WWTF during the summer is estimated to be approximately 700 kg/day. This suggests that the current oxygen supply is more than adequate for the required $CBOD_5$ removal.

2.3.5.2 Aeration Piping and Diffusers

The aeration piping and diffuser network is designed to provide the required oxygen supply to Cell No.1 and Cell No.2. The distribution of diffusers was done be considering each cell to have two (2) parts, and the header density was decreased in the direction of flow (since the oxygen demand is highest at the inlet). The following table summarizes the aerator design in the existing lagoon.

Table 2-2: Aeration Requirements Summary - Existing Facility

Design Condition	Total Daily 0_2 Required (kg)	Min. Number of Static Tubes	Static Tube Distribution	
			Cell No.1	Cell No.2
20-year (2015)	2237	266	200	66
Ultimate	2796	330	247	83

2.3.6 Ultraviolet (UV) Disinfection

In 2009-2010, the previously existing chlorination system was replaced with a new ultraviolet (UV) effluent disinfection system. This included the construction of a new +/-74 sq.m. UV building and flow metering chamber (Parshall flume with ultrasonic level sensor). Piping modifications were made to accommodate the new inlet and outlet piping, as well as by-pass piping in the event of high flows in excess of the UV system's capacity or to allow flows to be diverted to allow for maintenance.

The UV system consists of a single-channel, two (2)-bank Trojan UV3000Plus system, with each bank containing seven (7) UV modules of eight (8) lamps each. Therefore, a total of 112 UV lights are contained within the two (2) banks.

The UV system is designed based on the following characteristics:

- Peak Disinfection Flow Rate: 19,306 m^3/day (5.1 US MGD) @ 40% UVT
- Peak Hydraulic Flow Rate: 37,854 m^3/day (10.0 US MGD)
- Effluent standards to be achieved: Maximum 200 fecal coliform/100ml

The water level in the UV channel is controlled by an automatic level controller, which allows the level in the UV channel to remain relatively constant, while accommodating variations in flows.

2.3.7 Outfall

The existing WWTF treated effluent outfall is presently at the northern most part of the WWTF boundary and its current configuration is shown on Drawing 2-1 and Figure 2-2. During regular operation, the UV Disinfected effluent is discharged into a 280 m long narrow channel located at the northern most part of the GSSC Facility, shown in Figure 2-2 and Figure 2-3. Where the narrow trench does not have significant flow from any other source, it is referred to as an "open pipe" in accordance with the CCME guidelines with no mixing until it is discharged into the 3.7 ha shallow basin referred to as Lac des Boudreau Ouest (Figure 2-3) that is connected to the Northumberland Strait via a small shallow channel.

Figure 2-2: WWTF Discharge

Figure 2-3: Narrow Outfall Trench

Figure 2-3 – Outfall to Northumberland Strait

2.3.8 Backup Power

Currently there are two (2) backup power generators installed at the WWTF to provide power to critical systems in the event of a power failure. They are described as follows:

- Screw Pump Building: This generator was installed as part of an addition made to the screw pump building during the 1994 upgrade of the WWTF. This generator provides power to the screw pumps, submersible pumps, pre-treatment components and building systems.
- Blower Building: This generator provides power to the building systems only.

2.3.9 Maintenance Garage

Located at the south west corner of the site, directly adjacent to the screw pump and pre-treatment buildings, this building is used for storage and to house equipment for grounds maintenance (mowing and snow removal). Furthermore, this building houses a work bench and tool storage area.

2.4 Site Assessment of Existing Infrastructure

On November 14, 2017 Crandall performed a visual assessment of existing above-ground facilities at the Cap-Brulé WWTF with Mr. Joey Frenette, B.Sc.,PTech – General Manager. Additional photos can be found in Appendix D From this assessment, a list of current deficiencies was compiled as follows:

- <u>Influent Pumping Station</u>
  - Building envelope: Siding appears to be original to the building construction and is generally in poor condition. Repair or replacement work will be required in the short term.
  - Roof: The roof is asphalt shingle construction and was replaced in 2014. It appears to be in good condition.
  - Wet-well: The existing concrete wet well for the screw pumps was noted as being in a state of significant distress and appears to be nearing the end of its useful life. Cracking was noted throughout, with localized areas of missing concrete and exposed reinforcement.
  - Screw Pumps: A recent upgrade was completed to the screw pumps, including replacement of two (2) of the three (3) screw pumps and repairs made to the channels to reduce screw bypass.
  - Submersible Pumps: This station was constructed in 2004 and appears to be in good condition. No deficiencies were noted.
  - Metering: While the metering components (flume with ultrasonic level sensor) were not inspected, it was noted that renewal of these components should be scheduled in the short term.

- **Pre-Treatment Building**
  - Envelope: No deficiencies noted
  - Roof: No deficiencies noted
  - Mechanical screen: The operator noted that it appears as though the screen allows a significant amount of solids through and believes smaller openings would be preferred. Plastic removal is also needed.
  - Grit removal system: No deficiencies noted. Equipment was noted as performing adequately with no major maintenance issues.
  - General
    - Humidity issues: The operator noted that internal humidity has always been a problem and the current HVAC systems are not capable of keeping up. When outside temperatures allow, the overhead door is routinely kept open to improve air circulation.
    - Structure: The operator noted that he believes a structural assessment should be completed on the structure due to the ongoing humidity issues.
- **Blower Building**
  - Envelope: No deficiencies noted
  - Roof: The roof is asphalt shingle construction and was replaced in 2014. It appears to be in good condition.
  - Blower components: no deficiencies noted.
- **Maintenance Garage**
  - Envelope: Siding appears to be original to the building construction and is generally in poor condition. Repair or replacement work will be required in the short term.
  - Roof: No deficiencies noted.
  - General:
    - The operator noted that they need more storage space than is currently available.

- UV Building
  - Envelope: No deficiencies noted
  - Roof: No deficiencies noted
  - UV equipment: No deficiencies noted
  - Ventilation: There are humidity issues in the building during the summer that require the operator to leave the door open.
  - General:
    - Constructed in 2009. Appears to be in very good condition.
- Lagoon Cells
  - Berms: Recent settlement have been noted around Cell No. 2. While this is not currently impacting operations, further investigation is recommended to diagnose the settlement.

2.4.1 Remaining Life of Major Components

To provide an indication of when the existing major components at the treatment plant will require renewal, an assessment of the age and expected useful life of each component was completed. While age is not the only factor in a component's condition, it can provide a reasonable estimation of when an asset requires replacement. The following table summarizes this analysis:

Table 2-3: Existing WWTF Components – Expected Remaining Life

Component Description	Date of Construction (Component Age)	Expected Useful Life	Expected Remaining Life
Influent Pumping Station			
- Building Envelope (original)	1971 (47)	40	-7
- Building Envelope (addition)	1995 (23)	40	17
- Building Roof	2014 (4)	20	16
- Building Structure (addition)	1995 (23)	75	52
- Building Structure (original)	1970 (48)	75	27
- Screw Pumps (2)	2012 (6)	25	19
- Screw Pump (1)	1971 (47)	25	-22
- Wet Well	1970 (48)	50	2
- Generator	1995 (23)	40	17
Pre-Treatment Building			
- Building Envelope	1995 (23)	40	17

Component Description	Date of Construction (Component Age)	Expected Useful Life	Expected Remaining Life
- Building Roof	1995 (23)	20	-3
- Building Structure	1995 (23)	75	52
- Bar Screen	1995 (23)	25	2
- Grit Well	1995 (23)	40	17
- Grit Extraction/Dewatering	1995 (23)	25	2
Maintenance Garage			
- Building Envelope	1971 (47)	40	-7
- Building Roof	1995 (23)	20	-3
- Building Structure	1971 (47)	75	18
Blower Building			
- Building Envelope	1995 (23)	40	17
- Building Roof	2014 (4)	20	16
- Building Structure	1995 (23)	75	52
- Blowers	1995 (23)	15	-8
- Generator	1995 (23)	40	17
UV Building			
- Building Envelope	2009 (9)	75	66
- Building Roof	2002 (9)	20	11
- Building Structure	2009 (9)	75	66
- UV Lamps	2009 (9)	25	16
- Metering Chamber	2009 (9)	25	16
Lagoon Cell No.1			
- Aeration Piping	1995 (23)	25	2
- Liner	1995 (23)	50	27
Lagoon Cell No.2			
- Aeration Piping	1995 (23)	25	2
- Liner	1995 (23)	50	27
Lagoon Cell No.3			
- Clay Liner	1971 (47)	50	3

As shown above, many of the existing assets and major components at the wastewater treatment plant have reached or are nearing the end of their expected useful life. Therefore, regardless of any required upgrade to the WWTF to improve capacity, a major lifecycle renewal is anticipated in the short term (0-5 years).

2.5 Existing Design Criteria and Flows

2.5.1 Existing WWTF Design Criteria

The current lagoon was designed for the following design criteria during the WWTF upgrades completed in 1994.

- Design Period: 20 years (1995-2015) – [Based on Loading Projections]
- Design Population:
  - Base: 9,025
  - Summer Peak: 15,040
- Average Dry Weather Flow: 6,815 m^3/d (1.8 USmgd)
- Peak Wet Weather Flow: 26,650 m^3/d (7.04 USmgd)
- Influent BOD_5 176 mg/L
- Influent BOD_5 Loading/day 1,200 kg
- Influent TSS 140 mg/L
- Influent TSS Loading/day 960 kg
- Influent Phosphorous (P) 3 mg/L
- Influent TKN 20 mg/L
- Influent pH 7.0

2.5.2 Existing Service Population and Flow Rates

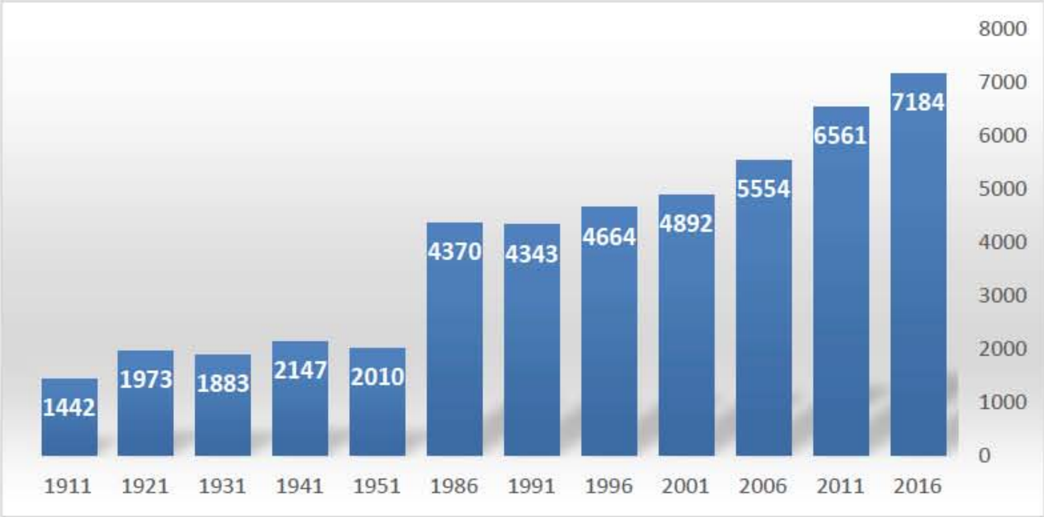
To establish the existing treatment plant's capacity to accommodate current and future flows and loading, the existing treatment conditions were reviewed in detail. This involved a review of available flow metering data, influent sampling data, and validation of flow data through theoretical estimates.

2.5.2.1 Service Population and Historical Growth

The Greater Shediac Sewerage Commission services the Town of Shediac and surrounding areas (including Shediac Cape, Cap Brulé & Pointe-du-Chêne), with a large seasonal variation in population. When confirming the existing population serviced by the GSSC, Canadian Census data for "Shediac –

Population Centre" was reviewed in detail, with the latest Census having been completed in 2016. The following graph summarizes historical census data as early as 1911.

Figure 2-4: Graph - Historical Census Population Data (Shediac Population Centre)



Using 2% growth rate on the 2016 Census data to estimate the current (2018) service population results in a figure of 7,475. Please see Section 3.1.1 for discussion on how the 2% growth rate was selected.

2.5.2.2 Theoretical Flows – Existing Conditions

Theoretical flow estimates were completed for the existing service area to compare theoretical flow estimates with metered flow data, to comment on metered flow patterns.

To estimate existing flows for various development types within the GSSC service boundary, the following assumptions were made:

Flow rate allowances (In accordance with the 2006 Atlantic Canada Wastewater Guidelines)

- Flow per Person: 320 L/person/day
- Commercial / Light Industrial 17 m^3/ha/day
- Campground 500 L/site/day
- Inflow and Infiltration
  - PVC Pipe 0.24 m^3/cm dia./km of pipe

- Concrete Pipe 0.48 m^3/cm dia./km of pipe

Average daily flow for residential areas was calculated by using Census data and the per person allowance above. For commercial, light industrial and institutional areas, existing flows were estimated through measurement of surface area currently occupied by development of this type (using recent aerial imagery and Town of Shediac zoning maps).

Inflow and Infiltration (I/I) flows were estimated through summation of existing GSSC sewer main by material and diameter using data from the GSSC master plan. An additional 15m of PVC service piping (100mm dia.) was assumed for each service connection

The existing average daily flows calculated for the GSSC service area are summarized as follows:

- Residential
  - Existing Population 7,475 (Estimated)
  - Flow/Person: 320 L/day
  - Average Daily Flow: 2,393 m^3/day
- Commercial / Light Industrial
  - Existing Developed Area: 189 ha
  - Flow per hectare: 17 m^3/day
  - Average Daily Flow: 3,212 m^3/day
- Tent/Trailer Camping
  - Site Count 990
  - Flow per Site: 500 L/day
  - Average Daily Flow: 495 m^3/day
- Inflow and Infiltration
  - Total pipe length 160 km
  - Inflow and Infiltration 1,010 m^3/day
- Total Average Daily Sanitary Flow 6,100 m^3/day
- Total Average Theoretical Daily Flow 7,134 m^3/day [Prior to adjustment, see Section 2.5.2.3]

Peak flow was estimated through calculating an equivalent population, by dividing the Total Average Daily Sanitary Flow by the residential flow allowance of 320 L/person/day. This gave an equivalent population of 19,839. Peak flow was then calculated by using the Harmon Equation shown below:

$$Pf = 1 + \frac{14}{4 + p^{0.5}}$$

Where:

Pf = Harmon Peaking Factor

p = Equivalent population in thousands

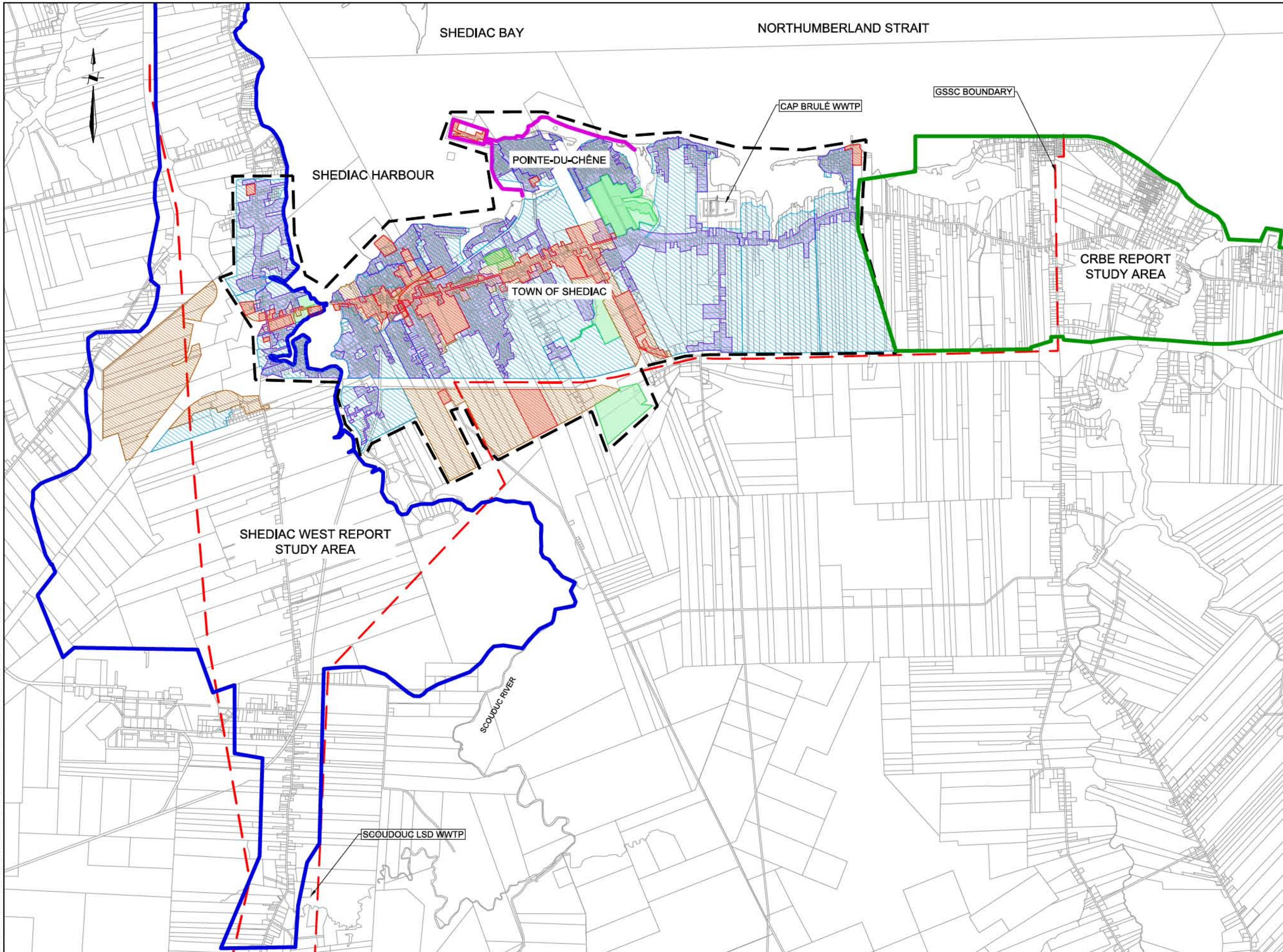
This gives an overall peaking factor for existing conditions of **2.65**, resulting in a theoretical peak flow of **17,870 m^3/day (4.72 MGD).**

Please see Drawing 2-3 on the following page for an overview of the existing and future GSSC sewershed areas.

2.5.2.3 Metered Flows

When available, metered flow data is often the preferred method for establishing design flows, both average daily and peak, for a wastewater facility. For the Cap-Brulé WWTF, there were several sources of flow metering data available through the commission's SCADA system. Those sources are as follows:

- **Pumping Station Data:** The Commission has a network of 24 pumping stations, and each station has either a flow meter or hour meter to record the flows leaving that station. While these stations do not capture the entire GSSC service area, they do capture a significant portion.
- **Influent Flow Meter – WWTF:** This flow meter captures much of the flow entering the WWTF, through the screw pump building (flow entering the station through LS24 and LS19 do not pass through this meter). This flow meter provides valuable data regarding the flow patterns entering the plant (before being buffered by the lagoon), but there are some concerns regarding the accuracy of the data due to the age of the equipment.



NOTES

EXISTING CONDITIONS

- EXISTING COMMERCIAL
- EXISTING RESIDENTIAL
- CAMP GROUND AREA
- GSSC BOUNDARY
- CAP BRULÉ WWTP FUTURE DEVELOPMENT AREA

FUTURE CONDITIONS

- FUTURE COMMERCIAL
- FUTURE RESIDENTIAL
- SHEDIAC WEST REPORT STUDY AREA
- CRBE REPORT STUDY AREA

NO.	DATE	REVISIONS	BY	APPR.
1.0	NOV 23/18	ISSUED FOR FINAL REPORT	GMG	SEB
0.0	JULY 10/18	ISSUED FOR DRAFT REPORT	DWD	SEB

GREATER SHEDIAC SEWERAGE COMMISSION
DES ÉGOUTS SHEDIAC ET BANLIEUES

crandall

PRELIMINARY ONLY
NOT TO BE USED FOR CONSTRUCTION

PROJECT TITLE

SHEDIAC EAST LONG-TERM WASTEWATER MANAGEMENT STRATEGY

SHEDIAC N.B.

DRAWING TITLE

GSSC SEWERSHED AREAS

Scale	Drawn By	Design By
250m 0 500m SCALE : 1:50 000	GMG	CS
	Checked By	Cadd Check
	SEB	BW
	Sheet 1 of 1	

File Name 2-3.DWG

Drawing No. 2-3

- UV Flow Meter – WWTF: This flow meter records all normal flows passing through the UV building. The readings from this flow meter are regularly checked by the Commission to validate the data. Data from this meter was used by the study team for average daily flows only, as peak flows are buffered by the lagoon.

These sources of data were used together for various elements of the study. However, a modified version of the hourly data from the influent flow meter was used most prevalently. As previously described, the hourly influent for meter data has the benefit of capturing the flow patterns at the WWTF before they are buffered by the lagoon cells. However, as shown in the graph on the following page and as previously discussed, there are concerns with the overall accuracy of the influent flow meter. As shown below, the daily average flows for the influent flow meter appear to generally be shifted upwards when compared to the UV flow meter. When comparing the average flow for each meter over the two (2) year data period, the discrepancy becomes more evident.

Table 2-4: Comparison of Metered Data - Influent vs. UV

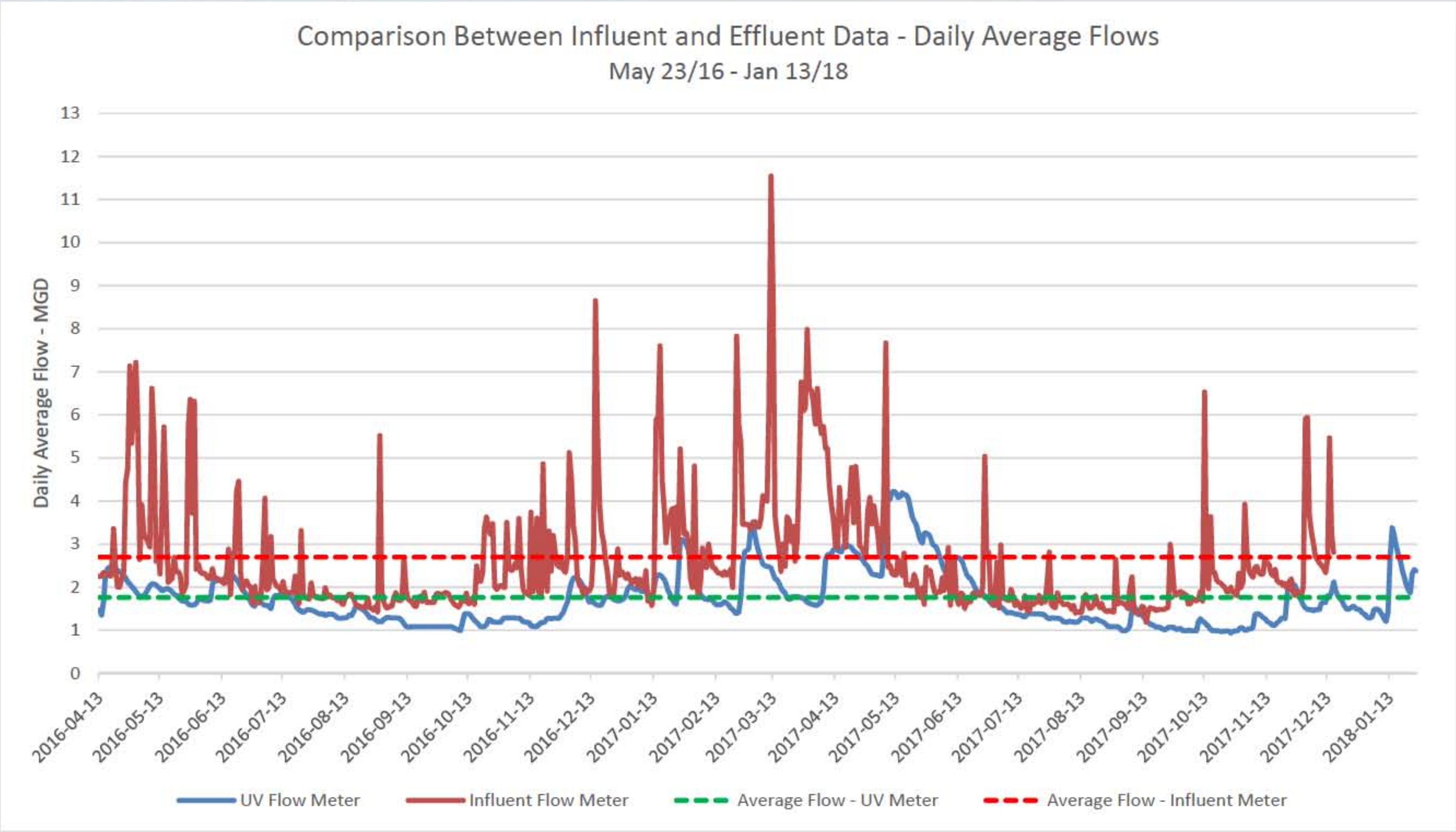
AVERAGE DAILY FLOW – INFLUENT	AVERAGE DAILY FLOW – UV METER	DIFFERENCE (INFLUENT / UV)
6,664 m^3/d (1.76 MGD)	10,219 m^3/d (2.70 MGD)	153%

It is proposed that the apparent shift in flow data between the two meters is a result of differences in metering technology.

Furthermore, through a review of the hydraulics through the WWTF (described in more detail in Section 2.4), there is the potential for bypass of the UV flow meter during high flow events. Due to the uncertainty of how much flow is unaccounted for by the UV flow meter, it was decided to adjust the metered values upward by 20% as a conservative approach. This resulted in an adjusted average daily flow of 7,997 m^3/d (2.12 MGD). The influent flow meter data was then modified so that the average daily flow aligned with the adjusted UV average daily flow. This modified data set was used throughout the study when referencing metered data.

It is recommended that the existing influent flow meter be reviewed in detail to determine the source of error. Prior to detailed design of any proposed upgrades to the WWTF, it is recommended that the flow assumptions made herein are reviewed accordingly

Figure 2-5: Graph - Comparison Between Influent and Effluent Meter Data



2.5.2.4 Selection of Existing Flow Conditions

To establish existing flow conditions for use in analysis of the existing WWTF and future upgrade scenarios, three (3) methods were used, including:

- Method 1: Evaluation of SCADA data at the existing pumping stations and calculating theoretical flows (as described in Section 2.5.2.2) for the area serviced by gravity.
- Method 2: Evaluation of SCADA data at the WWTF (both influent and effluent meters).
- Method 3: Theoretical calculation of flows from entire service area.

The resulting Average daily and Peak flows were calculated as follows:

Table 2-5: Existing Flow - Method Comparison

EXISTING FLOWS		
	Peak Flow (m^3/d)	Average Flow (m^3/d)
Method 1	27,803	7,501
Method 2	33,021[1]	7,997[2]
Method 3	17,872	7,359

1. The peak flow shown in the table above was established as the maximum pumping capacity entering the WWTF. This represents the combined capacity of the screw pumps, LS 24 and LS 19. The influent flow meter has some isolated extreme flow readings which are much higher than this value. However, 97% of hourly flow readings over a two (2) year period were below the value shown.
2. The average daily flow shown is the adjusted UV meter data as described in Section 2.5.2.3

While the calculated average daily flow amounts from the three (3) methods are all similar, it was noted that there is a large variation between the calculated peak flows. The noted peak flows for each method range from Method 3 (17,872 m^3/day) for a purely theoretical analysis method, to Method 2 (33,021 m^3/day) which is derived completely from metered data. Method 1, which includes a blend of metered data and theoretical estimates, lies between Method 1 and 2. It is proposed that this is due to the known issue of Inflow and Infiltration (I & I) in the GSSC system.

Following review of the various flow estimations, flows from Method 2 were selected to represent existing conditions. These flows were used to assess existing WWTF capacity as well as for estimating future flow conditions:

Table 2-6: Selected Existing Flows

	FLOW FROM METHOD 2 M^3/D [MGD]	SELECTED FLOW M^3/D [MGD]
Average Daily Flow	7,997 [2.11]	8,500 [2.25]
Peak Flow	33,021 [8.72]	33,100 [8.75]

2.5.3 Existing WWTF Loading

Treatment plant loading, in terms of Kg of $CBOD_5$ and TSS per day, is another critical parameter that was established for the purposes of evaluating various concepts and development scenarios.

Loading is a function of effluent concentration multiplied by the daily average flow. Typically, new municipal wastewater treatment plants are designed to a loading of 200/200 kg/day of $CBOD_5$/TSS respectively. However, due to the significant influence that I&I plays on the overall flow patterns at this facility, the loading from existing flow areas had to be accounted for separately.

The following graphs show the results of bi-weekly influent sampling at the Cap-Brulé WWTF for the 2016-2017 period:

Figure 2-6: Graph - Influent Sample - $CBOD_5$

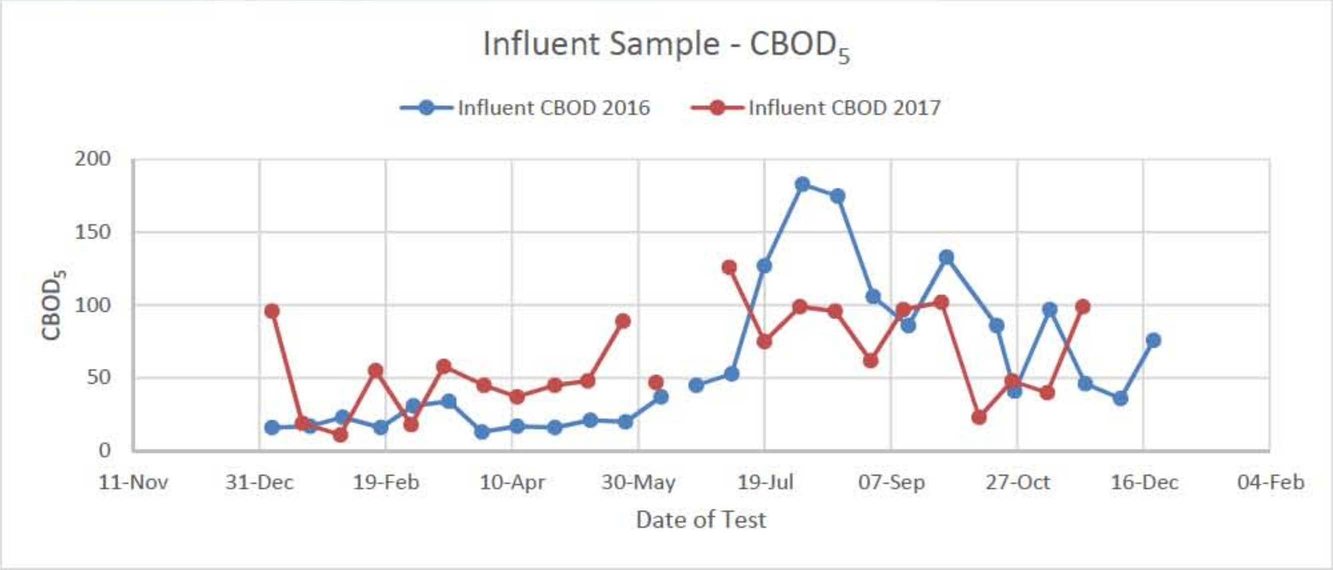
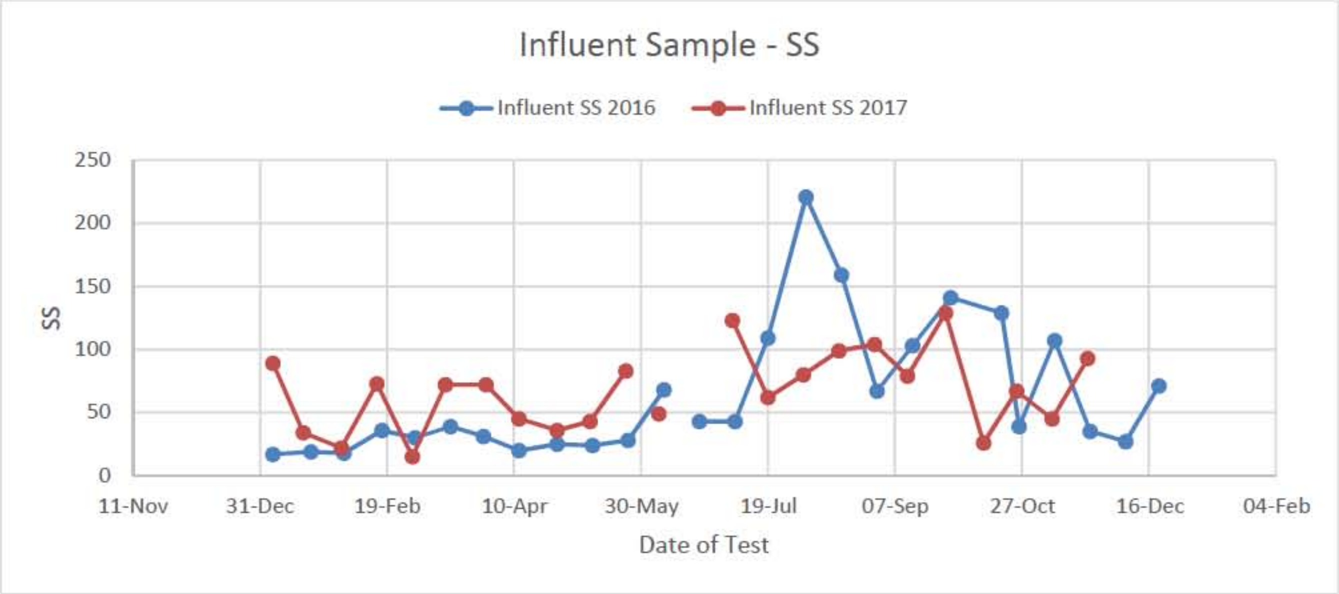


Figure 2-7: Graph - Influent Sample - TSS



While in late summer of 2016, sampling results were in the range of "typical" municipal effluent, average values on an annual basis were highly diluted. Averages over the 2-year sampling period were calculated as:

- Average BOD_5 concentration: 59 mg/l
- Average TSS concentration: 63 mg/l

This results in current average daily loading of 443 / 473 kg of BOD_5/TSS respectively.

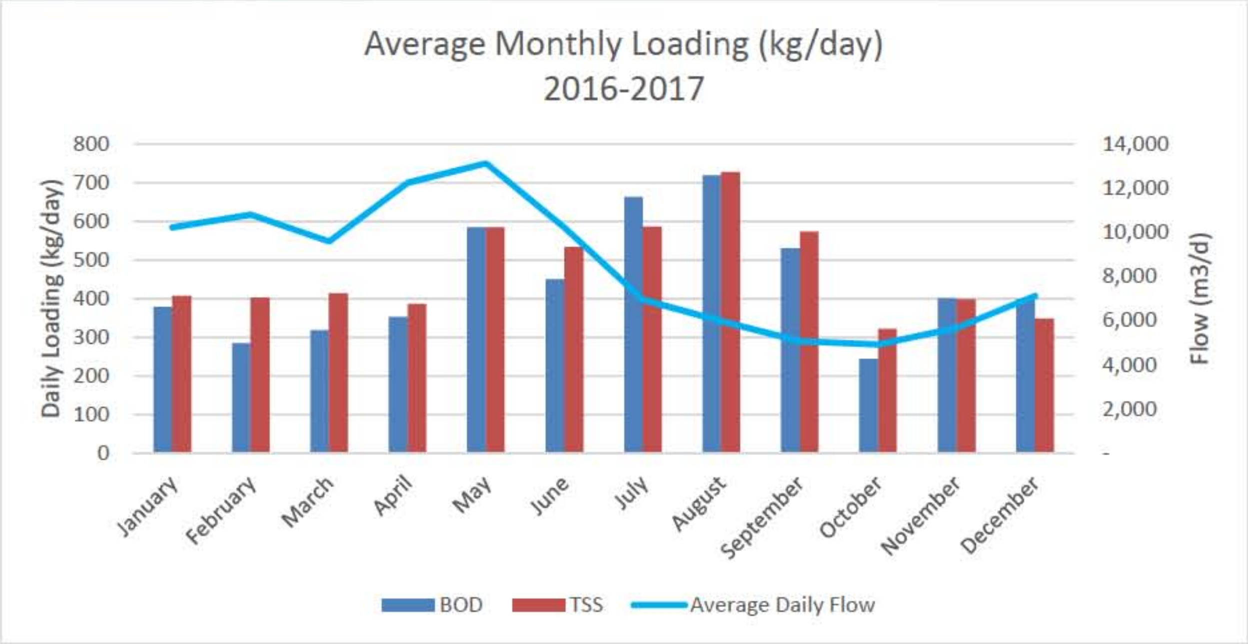
To account for the variation in flow rates and loading concentrations on a monthly basis, both the modified influent data set (See Section 2.5.2.4) and the average monthly influent concentrations shown previously were used to estimate the currently daily influent $CBOD_5$/TSS loading by month. The following graph summarizes the calculations:

Table 2-7: Existing Loading (BOD_5/TSS)

	BOD_5 (MG/L)			TSS (MG/L)			AVG MONTHLY FLOW (M3/D)	AVG MONTHLY EXISTING LOADING (KG/D)	
	'17	'16	Avg	'17	'16	Avg		BOD_5	TSS
January	17	58	37	18	62	40	10,220	378	406
February	20	33	26	27	48	37	10,805	284	402
March	26	40	33	33	53	43	9,586	318	414
April	17	41	29	23	41	32	12,255	352	386
May	21	69	45	26	63	45	13,128	584	584
June	41	47	44	56	49	52	10,217	450	534
July	90	101	95	76	93	84	6,956	663	586
August	155	86	120	149	94	122	5,977	718	727
September	110	100	105	122	104	113	5,069	530	573
October	64	36	50	84	47	65	4,925	244	321
November	72	70	71	71	69	70	5,682	401	398
December	56		56	49		49	7,121	399	349

As demonstrated in Table 2-7: Existing Loading (BOD_5/TSS) above and in the graph below, although average daily flow decreases significantly during the summer months (lower I/I contribution), loading increases substantially.

Figure 2-8: Graph - Monthly Avg. Flow VS. Loading



It is proposed that this is a result of the significant increase in population in the Town of Shediac & surrounding areas during summer months corresponding with a major reduction in I/I. The loading during summer months was noted as being between 1.5-1.6 times the average annual loading at the facility. This correlates with the estimated increase in population noted in previous reports prepared for the GSSC including the design brief for the latest WWTF upgrade in 1994.

Therefore, to account for the significant influence that the tourist season has on loading and flows at the Cap-Brulé WWTF, a factor of 1.5 times was applied to estimates of future development flows. This is discussed further in Section 3.1.3.

2.5.4 Inflow and Infiltration

The GSSC has been conducting inflow and infiltration studies throughout the sewer shed basin since 2001. A summary of past reports are as follows:

- **Phase I - Infiltration and Inflow Identification**

  Phase I report completed by Crandall Engineering and the sub-consultant Hydro-com Technologies was requested By the GSSC to evaluate the causes of the extreme sanitary flow conditions in 2001. Particular attention was made to areas of experiencing higher than normal lift station overflows; preliminary assessment of the trunk sewer, Drainage basin of Lift Station No. 3 and areas within Pointe-du-Chene.

- **Phase II - Infiltration and Inflow Identification**

  As part of the recommendations of Phase I, the Phase 2 (2002) report included additional flow analysis on areas of concern within the GSSC sewage basin. These areas were all the flows west of Lift Station No. 3 on Dock Street and a more detailed analysis on the existing Trunk sewer discharging to the WWTF on Cap- Brule Rd.

- **Phase III - Infiltration and Inflow Identification**

  It was recommended that the Phase III work be focused on areas identified as the highest I/I contributors under Phase II. As a result, the basins of lift stations 1, 2, 3, 4, 5, 6 and 14 examined with flow metering, lift station hour meter data and sewer video inspection.

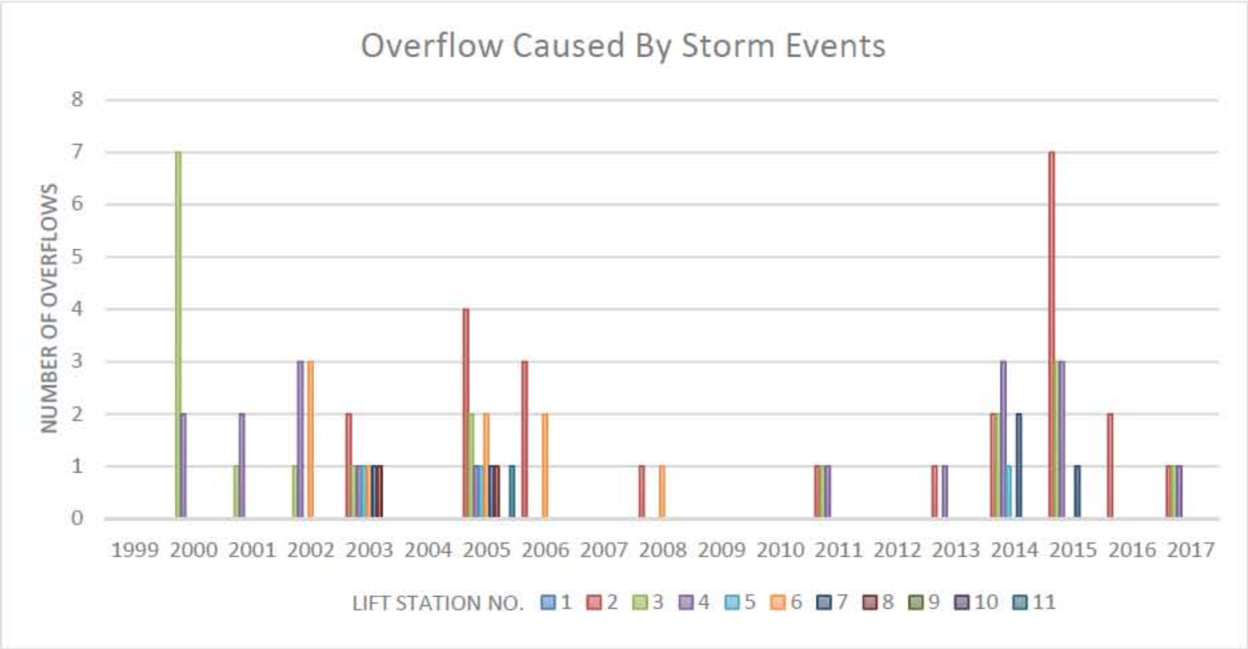
- **Phase IV and V – Inflow and Infiltration Identification**

  Phase IV and V were smaller localized areas of flow metering requested by the GSSC to review. These areas consisted of the Pointe-du-Chêne area, Scoudouc and the discharge locations of local camp grounds. The results were presented in small presentations at the public monthly GSSC meetings.

In order to reduce inflow and infiltration, however, a large effort to reduce cross connections (storm sewer connections) and sump pumps (private drain tile connections) will be required. If cross connections and sump pumps are not corrected it will not matter how much of the sewer infrastructure is upgraded or renewed as the main source of extraneous flows will still enter the system from private property. The reduction of I&I has a tremendous impact on the future treatment and collection of the GSSC system. High I&I impacts the hydraulics of a collection system and WWTF and reduces the hydraulic retention time of a lagoon system.

The following table illustrates the number of overflow events due to rain or snow melt from 2000 to present recorded per lift station effected within the GSSC System.

Figure 2-9: Graph - Overflow Counts by Year



The inflow and infiltration programs have been a key asset in diagnosing key problematic areas or areas of concern relating to extraneous flows entering the GSSC system that ultimately control the treatment efficiency at the WWTF. The program remains a continuing effort by the GSSC to reduce inflow and infiltration within the present system.

2.6 Existing Treatment Standards & Effluent Quality

The existing lagoon was designed (1994 upgrades) to provide the following effluent standards:

- $CBOD_5$ — 20 mg/L (May – October)
- TSS — 40 mg/L (May – October)
- Dissolved Oxygen — > 2mg/L
- Phosphorous (P) — 2 mg/L
- Nitrogen (as TKN) — 10 mg/L
- pH — 6.5 – 9.5

Since that time, an Environmental Risk Assessment was completed which reviewed the effluent quality objectives for the WWTF. The following sections describe the findings of that assessment and the resulting target treatment standards.

2.6.1 Environmental Risk Assessment

An Environmental Risk Assessment (ERA) was completed for the Cap-Brulé WWTF in 2014. The purpose of the ERA was to determine, based on current conditions, if the existing WWTF effluent is negatively impacting the receiving environment, and to what degree is it is. In addition, the ERA process establishes site-specific effluent discharge objectives (EDOs), calculated as a function of the receiving water quality and flow, to ensure the ongoing protection of the receiving water.

Therefore, as a result of this assessment, various recommendations and conclusions were made, including a list of EDOs specific to the Cap-Brulé facility in its existing configuration. The results and recommendations are summarized below.

1. The GSSC's Cap-Brulé WWTF is classified as a "medium" wastewater treatment facility under the CCME guidelines, based on its average flow being greater than 2,500 m^3/day and less than 17,500 m^3/day.
2. Based on this classification, the CCME guidelines identify an extensive list of "Potential Substances of Concern", which were analysed in the WWTF effluent.

3. A field investigation of the area of the WWTF outfall was conducted to determine the receiving water characteristics, including width, depth, velocity, and pattern of flow, as well as background water quality characteristics. Dye tests were done to assess mixing achieved downstream of the effluent discharge point. Poor mixing conditions were found in the receiving water; although the outfall was originally free-flowing when it was constructed, tidal influence and changes in the sand dunes over time have isolated the effluent discharge location, essentially creating a pond. Therefore, further studies were recommended regarding a new effluent discharge pipe location (Refer to Section 3.5 for further discussion).
4. It was determined that the Cap-Brulé WWTF effluent is meeting the CCME and NBDELG requirements for TSS and $CBOD_5$.
5. Fish toxicity tests were carried out on the effluent at various concentrations to determine any short- and long-term effects on aquatic life. Based on the CCME requirements, the effluent is to be non-acutely toxic, while a chronic toxicity EDO was set at 1.8 TUc for this facility. In eight (8) acute toxicity tests, all except one (1) were non-lethal. In the quarterly chronic toxicity tests all except one (1) were acceptable. Follow-up tests were completed to confirm the subpar test results, and the results were acceptable. Regular toxicity monitoring completed since that time has also produced acceptable test results.
6. Environmental Quality Objectives were identified for the receiving water, and EDOs were determined for the effluent discharge. As the list of substances is lengthy, and it was found that most substances were not of concern for this facility, Table 2-8 summarizes the key EDOs for the current WWTF.

Table 2-8: Key EDOs for Existing Cap-Brulé WWTF

TEST GROUP	EDO
$CBOD_5$	25.0 mg/L
TSS	25.0 mg/L
Un-ionized Ammonia	1.25 mg/L
TAN	1.74 mg/L*
E. coli	200 MPN/100ml

*TAN was selected for on-going monitoring although no treatment is currently provided for this substance and the current Certificate of Approval to Operate does not include an effluent limit. In addition, this EDO value is based on the current discharge location and should be re-evaluated based on a re-located outfall.

7. An effluent monitoring plan was prepared to ensure the effluent does not exceed its identified limits. The GSSC is required to follow this plan in addition to their Certificate of Approval to Operate requirements.

2.6.2 Approval to Operate

The GSSC's Cap-Brulé WWTF is required to comply with the requirements of its NBDELG-issued Certificate of Approval to Operate (CAO). The current CAO, issued in 2014, states the following effluent standards that the facility is required to meet:

1) $CBOD_5$: 25 mg (quarterly average)
2) TSS: 25 mg/L (quarterly average)
3) un-ionized ammonia: 1.25 (maximum)

However, the CAO also has special provisions for lagoon systems which allows the average TSS concentration to exclude effluent samples taken during the month of July, August, September or October, if that result was greater than 25 mg/L.

In addition, because the facility's current and projected future average flows are between 2,500 m^3/day and 50,000 m^3/day, the CAO requires the GSSC to test the effluent periodically for acute lethality (currently required once per year based on the facility having had four [4] consecutive quarterly non-acutely lethal test results). The WSER requires that the effluent be not acutely lethal.

Following the completion of the ERA Study, an effluent monitoring program was developed, as the GSSC is required to monitor the substances that the ERA identified as requiring on-going monitoring. These site-specific EDO values would typically be considered as the effluent limits that the facility is required to meet (pending confirmation of the regulators). However, since in this case it is recommended that the outfall location will be modified in order to achieve improved mixing and move the outfall away from the shoreline, EDOs for key substances of potential concern were developed specifically for the future WWTF outfall conditions using an assumed 1:100 dilution ratio. It is proposed that these values be considered as the minimum effluent discharge objective to be met by an upgraded facility, upon approval of the regulators, as follows:

Table 2-9 EDOs for Key Potential Substances of Concern – New Outfall Location

SUBSTANCE	EDO
TSS	25 mg/L
$CBOD_5$	25 mg/L
TAN	29.8 mg/L
TKN	7.3 mg/L
TP	1.7 mg/L
E. *coli*	200 MPN/100ml

2.6.3 Effluent Quality Monitoring

In order to establish the current treatment levels achieved by the lagoon, influent and effluent sampling data was received from the GSSC for the years of 2016-2017. These results are summarized in the following Table which indicates the average values and overall removal efficiency for $CBOD_5$ and TSS:

Table 2-10: Summary of Sampling and Treatment Efficiency

	Influent (mg/L)		Effluent (mg/L)				Removal Efficiency (%)	
Date	$CBOD_5$	TSS	$CBOD_5$	TSS	TAN	E. Coli[1]	$CBOD_5$	TSS
January	37	40	8	10	10		78%	75%
February	26	37	7	7	10		72%	82%
March	33	43	9	9	9		72%	80%
April	29	32	8	15	8		74%	54%
May	45	45	7	16	7	37	85%	65%
June	44	52	11	26	7	54	76%	50%
July	95	84	8	11	17	3	91%	87%
August	120	122	7	24	31	19	94%	81%
September	105	113	9	24	26	15	91%	79%
October	50	65	6	7	24	44	88%	90%
November	71	70	6	8	21		91%	89%
December	56	49	8	13	16		86%	74%
AVERAGE	59	63	8	14	15	29	83%	76%

1 As per the facility's CAO, effluent disinfection and E. *coli* sampling is required from May 1st to October 31st of each year, but has been recommended by NBDELG to be year-round as of January 1st, 2018

The sampling results indicate that the facility is generally meeting the requirements of its CAO, although the effluent values occasionally reach the facility's TSS effluent limit during the spring and summer months based on the quarterly average. High values during the summer can be a result of algae blooms in the shallow polishing cell. The facility's CAO allows high TSS values to be excluded from quarterly average values during the months of July, August, September and October for this reason.

2.7 Existing WWTF Capacity

As previously noted, the existing lagoon was designed in 1994 to receive flows of up to 6,815 m^3/day at a BOD_5 concentration of 176 mg/L. The design was based on providing effluent BOD_5 and TSS concentrations of 20 mg/L and 40 mg/L respectively, from May to October. Treatment was not required during the winter months at that time. To treat pathogens, the facility has a UV disinfection unit that was built in 2009.

The current facility therefore is designed to provide $CBOD_5$, TSS, and E.coli treatment. The current regulatory requirements require treatment of $CBOD_5$ and TSS to a maximum effluent concentration of 25 mg/L each, a non-acutely lethal effluent, and E. coli treatment (maximum 200 E. coli/100ml).

Currently, the average flow is approaching the 1994 design flow rate, although the BOD_5 loading is lower than the 1994 projections. However, the current NBDELG and the CCME requirement is for year-round $CBOD_5$ and TSS removal, targeting a maximum effluent concentration of 25 mg/L each. Therefore, based on the requirement to provide year-round treatment, the facility is nearing its theoretical capacity. A comparison of existing vs. design values is provided below:

- Average Daily Flow:
  - Current: 7,997 m^3/d [Wet-weather flows]
  - Design: 6,815 m^3/d [May – October]
  - Capacity: 8,664 m^3/d [5-day retention time capacity]
- Average Daily Loading (BOD_5)
  - Current 718 kg/d [Peak Month]
  - Design: 1,200 kg/d [May – October]

In practice, the $CBOD_5$ concentration in the lagoon effluent is well below the limit of 25 mg/L set in the facility's CAO, with an average effluent $CBOD_5$ concentration of 8mg/L over the years of 2016 and 2017.

It is noted that although the facility is still meeting its regulatory TSS treatment objectives based on the average effluent values, the TSS level periodically exceeds the effluent limit during the summer months (exceedance is allowed between July and October according to the CAO).

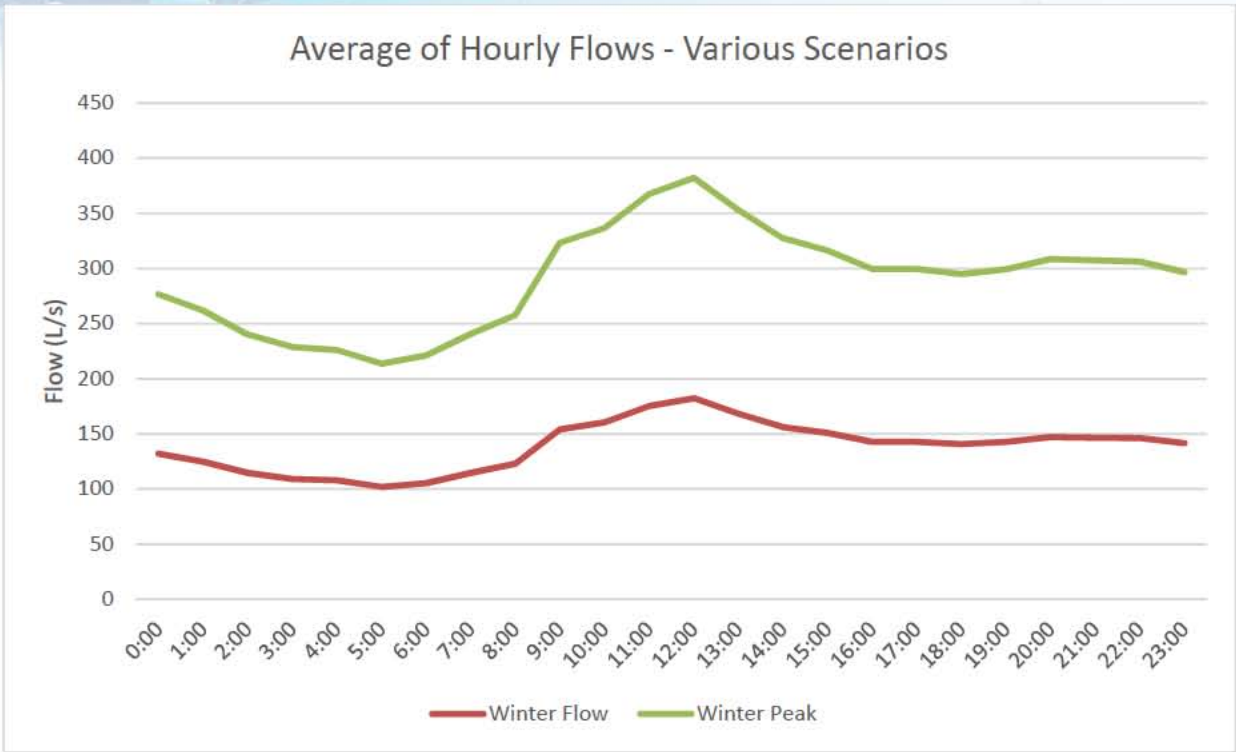
Since the facility is approaching its hydraulic and loading capacity, and since other major work is required to the outfall, as well as the aging buildings and aeration systems, it is expected that the facility will require upgrades to meet CAO objectives in the near future.

The above analysis is based on the lagoon portion of the treatment process and does not take into account the requirement for disinfection which is achieved through the UV disinfection system. The existing UV system is sized for a peak disinfection flow of 19,306 m^3/day (5.1 US MGD), which is below the 2018 peak flow. However, it is noted that the GSSC's regular sampling results have indicated that the facility is meeting its treatment objectives of 200 MPN / 100 mL of E. Coli from May 1st to October 31st of each year. Although the current UV system performance is acceptable based on meeting the current regulatory limits, UV system upgrades will be required in the future to provide sufficient capacity for the projected future flows.

2.8 Facility Hydraulics

To evaluate the existing facility's hydraulic adequacy, an assessment was completed of the hydraulic gradeline through the facility under various flow scenarios and Tailwater (tide) scenarios. This was done by preparing a hydraulic model of the WWTF from the discharge of the screw pumps to the outfall, in the SewerCAD software package. This model considers hydraulic losses through channels, flumes, bends, manholes, pre-treatment equipment, and piping.

Existing flow scenarios were based on influent flow data, which was analysed for both the summer and winter seasons to establish average daily patterns for the very different flow conditions. Furthermore, the average daily winter flow pattern was used to create a Peak winter flow scenario; the peak for which matches the selected existing peak flow from Table 2-6: Selected Existing Flows. The various flow scenarios are shown in the following graph.



Under the Winter Peak scenario, the model was set up to simulate the submersible pumps (LS 24) being activated between 9am and 4pm to address the peak flow period. This was done by dividing the flow entering the WWTF into flow at the screw pumps and flow directly into Cell No.3 (flow is typically bypassed here in a high flow event). It was found that if this bypass did not occur, the WWTF does not have the hydraulic capacity to accommodate all flow through the facility site piping and the pre-treatment building.

The following table summarizes the calculated water elevations at various key locations throughout the WWTF under the different flow scenarios:

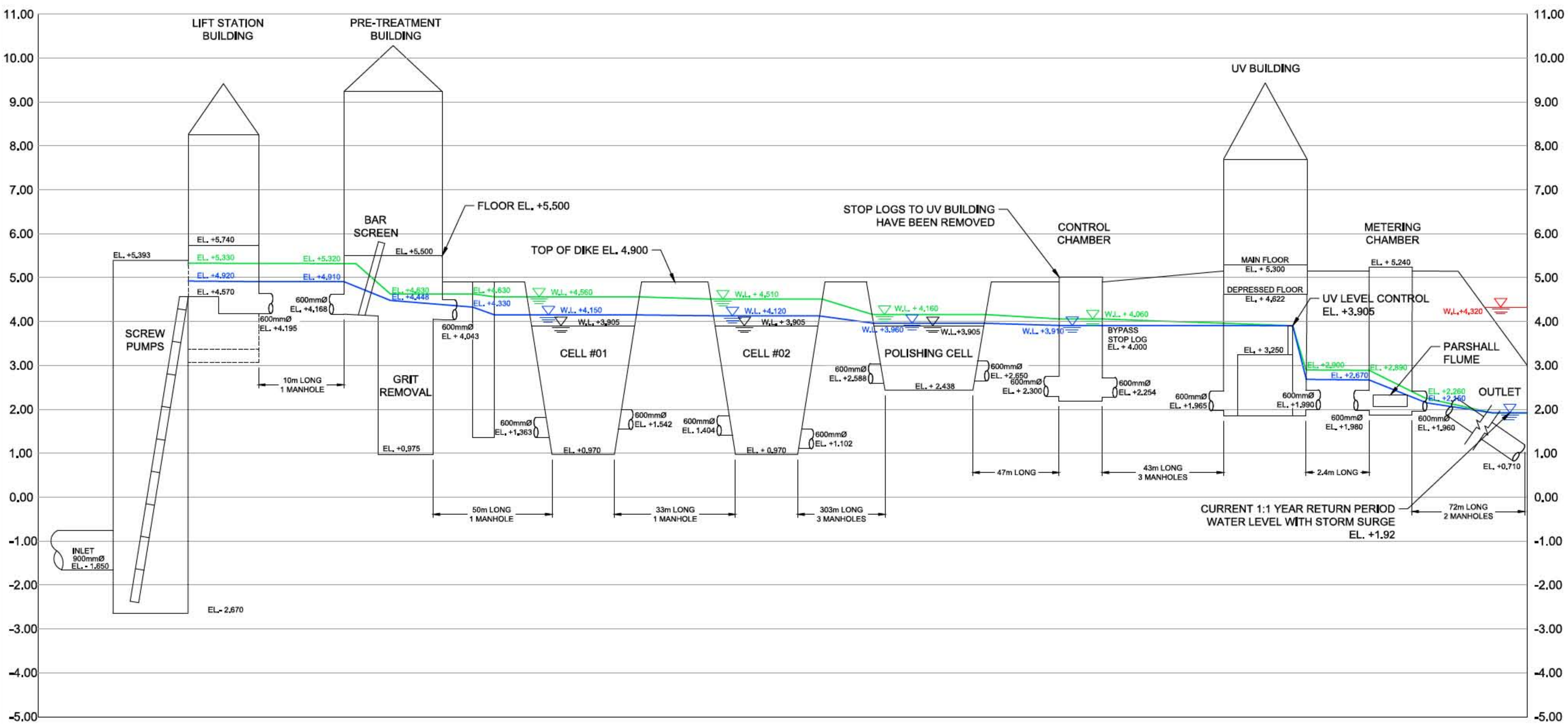
Table 2-11: Existing WWTF - Hydraulic Model Results

Flow Scenario	Cell No.1	Cell No.2	Cell No.3	UV Bypass	UV Channel
Winter Average	4.15	4.13	3.96	3.94	3.91
Winter Peak	4.56	4.54	4.16	4.06	3.99

When the hydraulic model was run under these flow scenarios, the following observations were made:

- During Average Winter flow conditions, the WWTF appears to generally perform well hydraulically with no major losses or bottlenecks noted in the plant. Water levels were noted to rise only moderately.
- During the Peak Winter flow scenario, there were significant hydraulic losses noted, particularly between Cell No.2 and the polishing cell.
- The Automatic Level Controller in the UV building has a significant influence on the overall water level in the WWTF.

Drawing 2-4 on the following page displays the calculated Hydraulic Grade Line (HGL) profile through the existing WWTF under each flow scenario.



NOTES

STANDING WATER
PEAK WL WINTER AVERAGE FLOW
PEAK WL WINTER PEAK FLOW
PEAK 1:100 YEAR WL YEAR 2100

NO.	DATE	REVISIONS	BY	APPR.
1.0	NOV 23/18	ISSUED FOR FINAL REPORT	GMG	SEB
0.0	JULY 10/18	ISSUED FOR DRAFT REPORT	DWD	SEB

GREATER SHEDIAC SEWERAGE COMMISSION
DES ÉGOUTS SHEDIAC ET BANLIEUES

crandall

PRELIMINARY ONLY
NOT TO BE USED FOR CONSTRUCTION

PROJECT TITLE

SHEDIAC EAST LONG-TERM WASTEWATER MANAGEMENT STRATEGY

LOCATION N.B.

DRAWING TITLE

EXISTING WWTF HYDRAULICS

Scale	Drawn By	Design By
N.T.S	DWD	SEB
	Checked By SEB	Cadd Check BW
	Sheet 1 of 1	

File Name 2-4.DWG

Drawing No. 2-4

2.9 Recommendations – Existing Conditions

Based on our analysis of existing conditions at the Cap-Brulé WWTF as described in the preceding sections, we have provided the following recommendations:

1. Complete flow monitoring at the WWTF to validate flow assumptions presented herein. This should be done as part of preliminary design activities for future upgrades.
2. Numerous components at the WWTF are nearing the end of their useful life and will require a lifecycle replacement in the short term (0-5 years). These components include:
    a. Pre-treatment equipment including the bar screen and grit removal systems;
    b. Pre-treatment building structure;
    c. Blowers and aeration piping;
    d. Screw pump wet-well and flow meter.
3. As previously recommended, a major upgrade of the outfall is required in the short term to address deficiencies with the effluent mixing rates in the current outfall.
4. The existing WWTF, while currently providing acceptable treatment levels in practice, is reaching the limits of its hydraulic capacity and TSS removal requirements.
5. The required upgrades to the facility in the short term (0-5 years) are significant and warrant consideration of the long-term requirements at the Cap-Brulé WWTF. It is therefore recommended that these component upgrades be included in a more comprehensive improvement to the WWTF to accommodate the next 25 years of design life. Please see the following sections for further discussion.

3 Long Term Planning

The following sections aim to establish probable treatment conditions for which an upgrade to the Cap-Brulé WWTF should be designed for. After those conditions were established, conceptual design was completed for treatment plant upgrade options.

3.1 Flow Conditions

3.1.1 Population Growth

To establish the current population (2018) and to estimate future population growth, historical growth rates from Census data were analysed.

Table 3-1: Historical Growth Rates - Shediac Population Centre

Year	Population	Growth Between Periods	Avg. Annual Growth Between Periods
1911	1442	-	-
1921	1973	531	3.18%
1931	1883	-90	-0.47%
1941	2147	264	1.32%
1951	2010	-137	-0.66%
1986	4370	2360	2.24%
1991	4343	-27	-0.12%
1996	4664	321	1.44%
2001	4892	228	0.96%
2006	5554	662	2.57%
2011	6561	1007	3.39%
2016	7184	623	1.83%

Historical growth rates for three (3) periods were calculated to review recent and long-term trends.

- 10-year period (2006 – 2016)
    - Population increase 1630
    - Average annual increase 2.61%

- 25-year period (1991 – 2016)
    - Population increase 2841
    - Average annual increase 2.03%
- 65-year period (1951 – 2016)
    - Population increase: 5174
    - Average annual increase: 1.98%

Based on this analysis, it would appear as though a growth rate of between 2% - 3% is reflective of the study area. These values were compared to the following sources:

- *50-year development plan for the West Portion of the Commission Service Area*: this study references demographic predictions presented in a Federally funded study on climate change in the Shediac Bay area:
    - Optimistic Projection: 1.8% growth per year
    - Pessimistic Projection: 0.8% growth per year
- *Town of Shediac – Water Distribution Master Plan, 2014*: this study was prepared for the Town for long term planning of potable water distribution. To estimate future system demands, an analysis of Census data was completed. An average annual future growth rate of 2% was proposed.

As a result of this analysis, a conservative (optimistic) **long term growth model of 2% per year was selected for the 50-year scope of this study.** The resulting population projection for the Town of Shediac are shown in the following table:

Table 3-2: Projected Population - 50-year Period

Year	Population	Growth Between periods	Avg. Annual Growth Between periods
1991	4,343	-27	-0.12%
1996	4,664	321	1.44%
2001	4,892	228	0.96%
2006	5,554	662	2.57%
2011	6,561	1007	3.39%
2016	7,184	623	1.83%
2018 (Current)	7,475 *	291	2.00%
2043 (25 years)	12,277 *	4802	2.00%
2068 (50 years)	20,158 *	7881	2.00%

* Projected

These population projections were used to establish estimates of future flow and loading rates for the conceptual upgrades to the Cap-Brulé WWTF described herein.

3.1.2 Future Growth Areas

A review was completed of the potential growth areas in and adjacent to the Town of Shediac, to determine available land reserve and to comment on whether the projected population growth described in the previous section is possible within these growth areas. For the purposes of this analysis, growth areas were separated between those areas currently within GSSC's service boundary, and other areas. These areas were dealt with separately as described below and as shown on Drawing 2-3, in Section 2.5.

3.1.2.1 Community Rural de Beaubassin Est

This large area to the east of Shediac is currently not serviced by the Commission and does not currently have any municipal wastewater servicing. A study was commissioned in 2008 to review options for providing municipal servicing to this large rural area, and it was recommended that new lagoon(s) be developed to service the area. As a result, this area was not specifically considered when projecting long term flow conditions at the Cap-Brulé WWTF.

3.1.2.2 Shediac West

This includes a large area to the West of the Scoudouc River, only a small portion of which is serviced by the Commission. Similar to the CRBE area, a study was commissioned to assess the growth potential in this area and to provide recommendations on sanitary sewer collection and treatment. It was determined through the study that this area should be serviced by a total of four (4) WWTF on the west side of the Scoudouc River, and not brought into the Cap-Brulé sewershed. As a result, this area was not specifically considered when projecting long term flow conditions at the Cap-Brulé WWTF.

3.1.2.3 Infilling Areas

Infilling areas were delineated as those areas which are currently serviced by GSSC or that fall within GSSC's service boundary. These areas were assumed to be collected by GSSC and brought to the Cap-Brulé WWTF.

Although the selected growth area does not specifically account for the CRBE or Shediac West areas, it is worth noting that projections of future flows were based on an annual growth rate for what is considered the Shediac Population Centre. Without being able to accurately predict where development in the Shediac area is likely to occur, this method predicts an overall increase in population to the Shediac area based on historical growth patterns. As a result, regardless of whether development occurs within the current Town limits, or in the other development areas noted, the WWTF concepts were sized to accommodate the anticipated growth.

3.1.3 Flow and Loading Projections

Future flow conditions for a 25 and 50-year development period were estimated at the WWTF to select and size various treatment plant upgrade alternatives. This was done by using the selected Existing flow conditions as presented in Table 2-6: Selected Existing Flows and Table 2-7: Existing Loading (BOD5/TSS) and grown according to the population projections presented in Section 3.1.1 Population Growth. The following assumptions were made:

- For non-residential areas development growth occurs at the same rate as population growth
- Development will occur at a similar density to existing development, for estimation of I/I.
- New development will produce flow at "standard" theoretical rates as described in Section 2.5.2.2.
- New development will meet current design and construction standards and have an associated low inflow/infiltration rate (low end of theoretical range for PVC).
- New development will produce standard effluent concentrations of 180 mg/l of $CBOD_5$ and 180 mg/l of TSS.
- Future flows and associated future loading during peak summer months (July and August) were increased by a factor of 1.5x to account for the impact of tourism and seasonal residents.

The following table summarizes the future influent conditions used to complete conceptual design of the various WWTF upgrade options presented later in this report.

Table 3-3: Future Design Flows and Loading Rates

	EXISTING	25-YEAR					50-YEAR				
	Flow	Flow	$CBOD_5$		TSS		Flow	$CBOD_5$		TSS	
	m^3/d	m^3/d	kg/d	mg/l	kg/d	mg/l	m^3/d	kg/d	mg/l	kg/d	mg/l
January	10,220	13,851	1,032	74	1,060	77	20,905	2,302	110	2,330	111
February	10,805	14,436	937	65	1,056	73	21,491	2,207	103	2,326	108
March	9,586	13,217	972	74	1,067	81	20,272	2,241	111	2,337	115
April	12,255	15,886	1,006	63	1,040	65	22,940	2,276	99	2,309	101
May	13,128	16,759	1,238	74	1,238	74	23,814	2,508	105	2,508	105
June	10,217	13,848	1,103	80	1,187	86	20,902	2,373	114	2,457	118
July	6,956	12,403	1,643	132	1,566	126	22,984	3,548	154	3,471	151
August	5,977	11,423	1,699	149	1,708	149	22,005	3,603	164	3,612	164
September	5,069	8,700	1,183	136	1,226	141	15,755	2,453	156	2,496	158
October	4,925	8,556	897	105	975	114	15,611	2,167	139	2,245	144
November	5,682	9,313	1,054	113	1,051	113	16,368	2,324	142	2,321	142
December	7,121	10,752	1,052	98	1,002	93	17,806	2,322	130	2,272	128
Average	8,495	12,429	1,151	93	1,181	95	20,071	2,527	126	2,557	127
Peak	33,021	41,855					55,542				

As shown in the table above, average flows are anticipated to increase by approximately 46% in 25 years and 136% in 50 years. Peak flows are expected to increase for the two scenarios by 27% and 68% respectively.

3.1.4 Impacts of I/I Reduction Programs

As noted previously, the flows entering the current WWTF are influenced considerably by Inflow and Infiltration in the collection system. This impacts a number of factors including a reduction in hydraulic retention time, treatment efficiency, hydraulic capacity, and pumping energy among others. A continued effort towards reduction of I/I in the GSSC collection system is recommended to extend the life of current and future infrastructure, and to provide a more stable operating condition at the WWTF.

3.2 Siting Considerations

The existing Cap-Brulé regional wastewater treatment facility was originally constructed in 1972 and the current location has served the commission well since that time. Considering continued development patterns in Greater Shediac, regulatory conditions, and the sensitive nature of the nearby Parlee Beach, a cursory review of alternative locations was completed.

When reviewing a candidate location, several criteria were considered including:

- Availability of Land
- Relative cost of collection system reconfiguration
- Increased Operations Cost
- Sensitivity of Receiving Environment

Please see Drawing 3-1 on the following page for an overview of the sites considered.

3.2.1 Option 1: Present WWTF Site

The existing WWTF was situated in its present location because of the surrounding area and location of development at that time. Located east of Cape Brule Road and the majority of the sewer service areas as shown on Drawing 2-3. The WWTF is also just south of the Northumberland Strait, providing an effective location for discharge of treated effluent.

Since construction, a significant amount of development has continued in the adjacent areas although mostly in the central area of Shediac. This has resulted in a limited amount of space for the WWTF to expand if required for future upgrading. Provincial environmental setbacks from protected areas also contribute to reduce land for upgrading.

3.2.2 Option 2: Alternate Site Inland

The approximate location of a potential alternate WWTF location is shown on Drawing 3-1. The concept would be to bring it further inland, South of Route 133, for access to more land. This area contains a large amount of undeveloped land that could potentially be purchased for a new WWTF location.

Additional Costs associated with moving the present location would be as follows;

1. A new Major Lift Station would be required at the outfall of the existing trunk sewer as the alternate location is geographically higher in elevation then the present WWTF location. A gravity fed WWTF would not be possible;
2. With the new major lift station, a large diameter force main would be required in order to pump the flows from the gravity sewer to the alternate WWTF location;
3. The outfall would be significantly longer in order to reach 350m into the Northumberland strait for adequate mixing;
4. Land cost, Legal and new easements;
5. Decommissioning of the old WWTF;
6. Clearing, grubbing, site works and access roads;
7. New well water system and infrastructure.

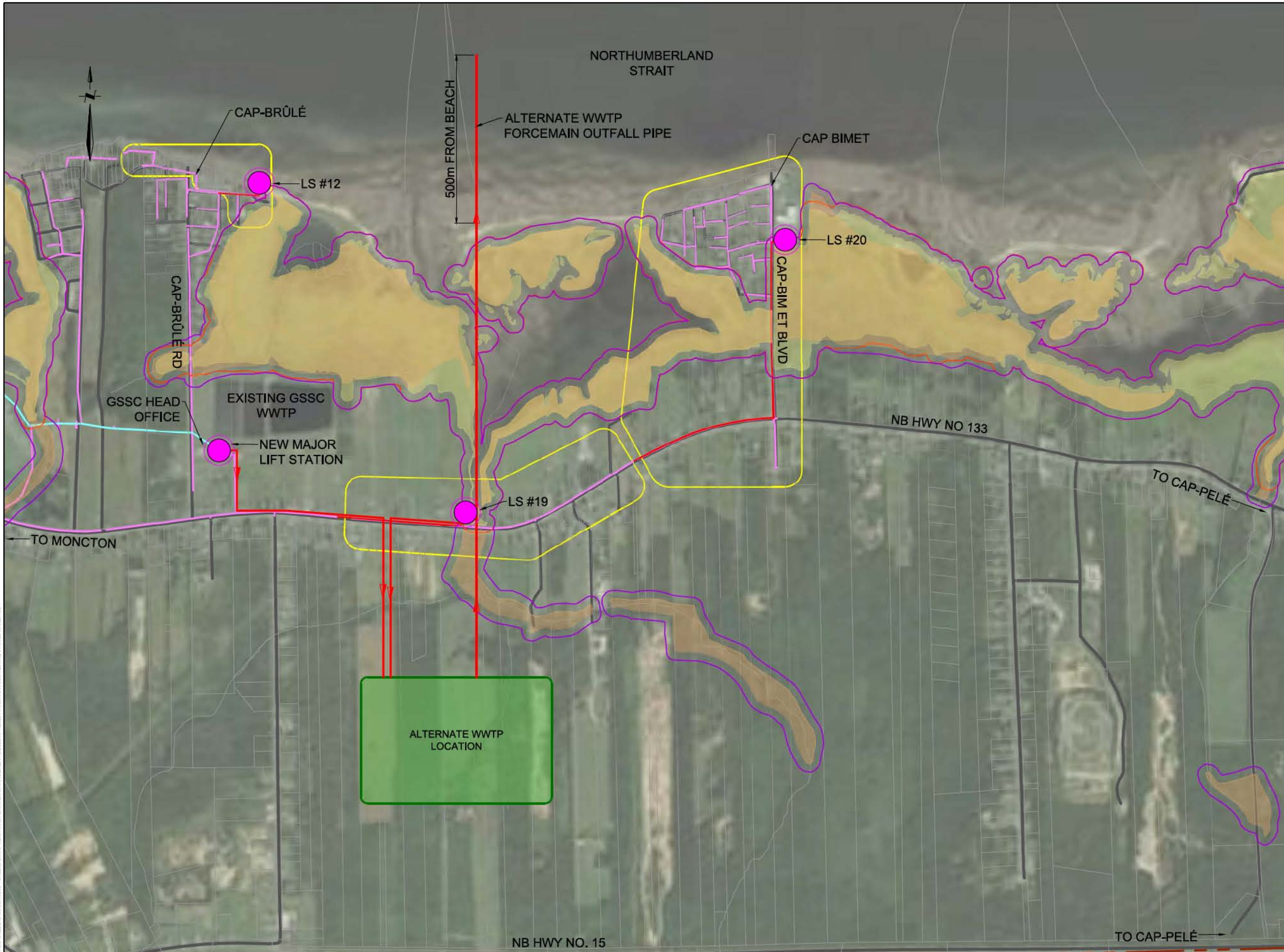
Major investments would be required for the reconfiguration for this option above, not including the new WWTF itself, As a result, this option was not considered to be viable.

3.2.3 Recommendation

Although other alternate WWTF locations were reviewed, such as along the Scoudouc River, these were not reviewed in detail as they would require either a significant Lift Station and forcemain to cross the Town of Shediac from the present WWTF or a significant reversal of gravity sewer main throughout the Town. The cost to move the WWTF anywhere to the west of the current location is not considered to be economical at this stage of the GSSC's development.

As a result, it is recommended that the WWTF remain in its present location for several reasons including;

1. There would be a significant added cost to reconfigure the current collection system infrastructure to accommodate any other location;
2. It is anticipated that the present WWTF location has enough available land to accommodate the future proposed upgrades. Depending on the selected option, some minor land purchase may be required;
3. The present location has the closest outfall distance to the Northumberland Strait as all other coastal areas are developed or environmentally protected;
4. New easements and environmental requirements would be needed for a new site and infrastructure.
5. A new site would require decommissioning of the old WWTF.



NOTES

- PROVINCIALLY SIGNIFICANT WETLAND (PSW)
- REGULATED WETLANDS MAP (RWM)
- PSW - 30m BUFFER
- RWM - 30m BUFFER

NO.	DATE	REVISIONS	BY	APPR.
1.0	NOV 23/18	ISSUED FOR FINAL REPORT	GMG	SEB
0.0	JULY 10/18	ISSUED FOR DRAFT REPORT	DWD	SEB

GREATER SHEDIAC SEWERAGE COMMISSION DES ÉGOUTS SHEDIAC ET BANLIEUES

crandall

PRELIMINARY ONLY
NOT TO BE USED FOR CONSTRUCTION

PROJECT TITLE

SHEDIAC EAST LONG-TERM WASTEWATER MANAGEMENT STRATEGY

SHEDIAC N.B.

DRAWING TITLE

ALTERNATE WWTF SITE LOCATION

Scale	Drawn By	Design By
N.T.S.	JHK	CG
	Checked By: CG	Cadd Check: BW
	Sheet 1 of 1	

File Name: 3-1.DWG

Drawing No. 3-1

3.3 Future Regulatory Conditions

3.3.1 Discussion with NBDELG

To identify any upcoming changes to the CCME treatment levels, Crandall staff had a conference call on December 1, 2017 with key staff at NBDELG. The Province directed the study team that any additional treatment objectives beyond those required as a result of the ERA are the prerogative of the GSSC. The Province was not aware of any changes to the CCME guidelines at the time of the conference call. It is anticipated however, that the recently requested requirement to disinfect year-round would remain as part of the next approval to operate for the Cap-Brulé facility.

3.3.2 Best Practice Review

Though the WWTF is currently meeting all the current NBDELG and CCME requirements, the commission may decide to treat their wastewater to a higher-than-required standard. Recognizing the significant value of the nearby Parlee Beach recreational area, Crandall has reviewed the implications of treating the following "Best Practice" effluent characteristics.

The following treatment facilities have been visited by Crandall staff as part of another project. These facilities were selected for review as they have put advanced treatment into place to protect sensitive receiving environments.

3.3.2.1 Sample Treatment Plant 1: Sault Ste. Marie, ON, BNR WWTF

The Sault Ste. Marie (SSM) facility is a BNR process facility and serves a design population of approximately 75,000, with a design capacity of 35,000 m^3/day. Therefore, it is slightly larger than 1.5 times the size the GSSC's WWTF will be, based on hydraulic capacity. The BNR plant has been operating for over 12 years and the facility is typically providing the following results:

- $CBOD_5$: 1 – 5 mg/L
- TSS: 1 – 5 mg/L
- Phosphorous 0.5 mg/L
- Ammonia nitrogen: 3 – 5 mg/L

With an average influent BOD_5 of 150-200 mg/L, this achieves a BOD_5 reduction of 96 – 97.5%.

The SSM facility has stand-by power for full plant capacity. Their sludge treatment consists of sending waste activated sludge to an aerated holding tank. A polymer is added to enhance dewatering and the sludge is sent to a DAF (dissolved air floatation) unit and the skimmed biosolids are sent to a centrifuge which results in 20 – 24% solids. The sludge cake is disposed of at the local landfill. Disinfection is by a UV (ultraviolet) system. With the more complex treatment process, it is noted that a higher level of operator knowledge and attention to plant operation is required, compared to a lagoon-style system.

3.3.2.2 Sample Treatment Plant 2: Dominion, NS, SBR WWTF

The Dominion WWTF is designed as a sequential batch reactor (SBR) process facility, operating since 2009. While SBR facilities are normally "fill and draw" batch processes, the Dominion WWTF is different. Its process is the Xylem Sanitaire ICEAS process, where there is continuous flow through a single long and narrow tank that performs aeration, settling and sludge removal operations. While this facility is not operated to achieve nutrient reduction, the process supplier indicated that its process could achieve this if a non-aerated tank were added to the inlet end of the tank.

The WWTF is designed for 4,000-5,000 persons, and a normal wastewater flow of 4,000 m^3/day. While the Dominion WWTF achieves effluent results of 6 mg/L $CBOD_5$ and 3 mg/L TSS, it has a weak raw wastewater from high I/I with an average influent $CBOD_5$ of 44.5 mg/L based on the information provided by the operator. This is equal to an 86% $CBOD_5$ removal efficiency. Pre-treatment consists of screening, grit removal and comminution (grinding); it does not have primary clarifiers. When the flows exceed a certain rate in this facility, it is necessary to by-pass the excess flows to avoid washing out the biological process. This facility also uses UV disinfection; E-coli readings are typically ~30 MPN/100 mL.

3.3.2.3 Sample Treatment Plant 3: Summerside, PEI, BNR WWTF

The Summerside WWTF is a BNR process facility, using the Modified Johannesburg process variation. This process modification uses seven (7) zones in each Bioreactor. It has primary clarifiers which were left from the original primary treatment plant at the same site. The WWTF was designed for a population of 18,175 and normal dry weather flow of 11,675 m^3/day, with a maximum hydraulic flow rate of 39,000

m^3/day. They have a much higher strength wastewater than GSSC, with influent $CBOD_5$ of 260 mg/L and TSS of 180 mg/L. The BNR facility has been in operation since 2008. The facility has a septage receiving station but the septage does not go through the BNR process. It is blended with the waste activated sludge after grit removal and processed with the sludge.

Their pre-treatment consists of screening, grit removal and comminution. Some of the sludge removed in the primary clarifiers has to be added back to the system later in the process to provide sufficient organic material to support the process. The return activated sludge (RAS) from the secondary clarifiers to the inlet of the Bioreactors is an important step to provide a high organic carbon environment. They also use UV disinfection. The facility has a small lab area where they do some testing to monitor the type of bacteria forming in the Bioreactor (microscopic assessment in the on-site lab) to ensure process control.

Summerside has an extensive sludge processing facility because they have an agreement with an agricultural products company to collect and distribute the end product. The waste activated sludge (and septage) goes through a sludge thickener followed by a sludge press to achieve 20 – 25% solids. This material is mixed with lime and passes through an oil-fired rotary kiln that dries the cake to 60% solids and kills bacteria. The plant is operated through a SCADA system that also incorporates the City's main wastewater Pumping Stations.

The following table summarizes the major features of each of the above facilities:

Table 3-4: Summary of Best Practice Site Review

	Capacity (M^3/day)	Community Pop	BOD	TSS	TAN (mg/L)	Phosphorus (mg/L)
Sault Ste Marie	35,000	75,000	25*	25*	3-5**	0.5**
Dominion	4,000	5,000	25*	25*	NO	NO
Summerside	11,675	18,175	10	10	5	1.0

*Generic WSER requirement

**Design parameter unavailable; actual treatment level being achieved is presented.

3.3.3 Proposed Future Treatment Objectives

Based on the discussion above, Crandall has considered two (2) scenarios for future treatment objectives, as follows:

- Scenario 1: Status Quo: This option includes treating to the known minimum quality objectives as outlined in the facility's Certificate of Approval to Operate as described in Section 2.6.2.
- Scenario 2: Best Practice : This option would involve treating to a higher-than-required standard, in recognition of the sensitivity of the receiving environment (Parlee Beach).

  As suggested by the best practice review carried out in Section 3.3.2, the current trend for larger sized WWTF's is to provide treatment for nutrients such as nitrogen and phosphorus. These substances are natural components found in all watercourses. However, when present in high concentrations these substances can lead to excess algae growth, called algal bloom, which can have toxic effects on marine species and significantly reduces the dissolved oxygen levels in the watercourse when the excess algae eventually decompose.

  If this Option were selected, the following additional or modified treatment objectives are recommended:

  - $CBOD_5$: 25 mg/L
  - TSS: 25 mg/L
  - Un-ionized Ammonia: 1.25 mg/L
  - TAN: 5.0 mg/L (based on best practice review)
  - TP: 1.0 mg/L (based on best practice review)
  - E.coli: 200 MPN / 100 mL

In Section 3.6, an evaluation of available treatment technologies to meet the treatment objectives of Scenario 2 is presented.

3.4 Impacts of Climate Change – Sea Level Rise

Whereas the Cap-Brulé WWTF is a coastally located facility and has the potential to be impacted by rising sea levels and/or storm surge, a desktop review was completed of potential impacts to the current and future facilities at this site.

While controversial, it is generally accepted in the scientific community that global climatic conditions are changing and are expected to continue to change into the future. For coastal areas, one of the primary concerns related to climate change is a relative change in sea levels caused by a combination of factors including:

- **Land Subsidence**: Also referred to as Vertical Land Motion, is the result of post-glacial adjustment of the earth's crust. While many areas in Canada are rising as a result of this effect, coastal areas are generally subsiding (lowering in elevation) compared to current levels.
- **Sea Level Rise**: Increasing global water levels are the result of a combination of factors including glacial meltwater, groundwater extraction, and changes to ocean currents.
- **Storm Surge**: Storm surges are the result of a reduction in atmospheric pressure and the wind associated with a storm. It is generally predicted that climate change is resulting in an increase in the severity and frequency of surge-producing storms.

In 2014, the report entitled *Updated Sea-Level Rise and Flooding Estimate for New Brunswick Coastal Sections* was submitted to the New Brunswick Department of Environment – Climate Change Secretariat by R.J. Daigle Enviro. This report is an update to a report published in 2012 by the same author to provide an estimation of Extreme Total Sea Levels for various planning periods and return-period events. The results presented for the Shediac area are summarized in Table 3-5:

Table 3-5: Estimated Extreme Sea Levels - Climate Change

Return Period (annual probability of occurrence) [1]	Estimated Extreme Total Sea-Levels (CGVD28)		
	Level 2010	Level 2030	Level 2100
1-Year (100%)	0.92 ± 0.2	1.78 ± 0.37	2.39 ± 0.68
10-Year (10%)	1.54 ± 0.2	2.40 ± 0.37	3.01 ± 0.68
100-Year (1%)	2.17 ± 0.2	3.03 ± 0.37	3.64 ± 0.68

1. Includes the influence of Storm Surge
2. Source: (R.J. Enviro, 2014) Table A-9 Zone 9: Westmorland County – County Line to Cape Spear

These values were used to perform a review of the WWTF for potential impacts and risks related to climate change.

3.4.2 Impacts on Existing Facilities

The existing WWTF was reviewed to identify risks related to climate change on current infrastructure, access and facility hydraulics. The following table summarizes the comparison of the elevations of key infrastructure to the Extreme Sea Levels (1:100-year return period) for 2030 and 2100.

Table 3-6: Summary of Sea Level Rise Impacts to WWTF

Location	Elevation (m)	2030 Elev. (m) 1:1 / 1:100	2030 Diff. (m) [1]	2100 Elev. (m) 1:1 / 1:100	2100 Diff. (m) [1]
Outfall	0.71	2.15 / 3.40	-2.69	3.07 / 4.32	-3.61
Metering Chamber	1.96		-1.44		-2.36
UV Channel	3.25		-0.15		-1.07
Top of Berms	Aprox. 4.9		1.5		0.58

1. Negative values denote a surcharged condition.

As shown in the table above, facility components following the UV building are at risk of being impacted by sea-levels during peak events. The potential risks for each component are:

- Outfall: Sea levels that exceed the outfall elevation have the potential to impact the hydraulic capacity of the WWTF discharge. However, due to the significant grade differential between the UV building and the discharge, the impact is expected to be minimal.

- Metering Chamber: When sea-levels exceed this elevation, there is a potential that flow metering accuracy could be impacted due to turbulence and backwater effects. Furthermore, corrosion of key components may be accelerated due to salt-water intrusion.
- UV Channel: Sea-levels in excess of this elevation have the potential to impact the water levels in the remainder of the WWTF. During extreme events, UV bypass could occur due to the impacted water levels.

While hydraulic functionality of the WWTF could be impacted by rising sea levels, it appears as though the risk of overtopping the lagoon berms is low. Furthermore, all facilities are well above the 1:100-year return period event in 2100.

Access to the WWTF appears to be unimpacted during an Extreme Sea-level event.

Drawing 3-2 on the following page shows the flooding limits for the 1:100-year return period event in the year 2100 (Elevation 4.32m)

3.4.3 Impacts to Future Upgrades

When considering future upgrades, the impacts of sea-level rise should be considered, particularly in the design of the required outfall improvements. As stated in Section 3.5, it is being recommended that the GSSC consider installing a pumped outfall, as opposed to a gravity outfall. One of the main drawbacks of a gravity outfall is that any change in sea-level could result in a direct increase in water levels in the WWTF. As a result, the berms would have to be raised considerably to accommodate the amount of hydraulic head required in addition to the large variation caused by tide and storm surge events.

3.5 WWTF Outfall Options

As a result of the ERA Study, where a new outfall location was recommended to meet the mixing requirements for this facility, a new location discharging directly to the Northumberland Strait was proposed. To construct a new outfall the following three (3) options were presented:

- Option 1: Gravity Outfall: This option involves installation of a new large diameter outfall. This option relies on water level at the lagoon to provide the required head to overcome friction losses.
- Option 2: Pumped Outfall: This involves construction of a new pumping station at the WWTF outfall to pump effluent to the proposed discharge location via a new forcemain.
- Option 3: Status Quo: This option would see the outfall remain in its current configuration, discharging into Lac des Boudreau Ouest. This option was not considered any further, as it does not meet the mixing requirements of CCME. Furthermore, it is expected that due to the nature of the current discharge, Lac des Boudreau Ouest could continue to be infilled by tidal action and sand deposition.

Drawing 3-3 on the following page shows the anticipated outfall concept.

3.5.1 Option 1: Gravity Outfall

A gravity outfall option was not evaluated in detail in the previous Outfall Study as it was not considered practical with the existing WWTF hydraulics and dike elevations. The gravity outfall option was reviewed again as part of this study and the following comment are provided:

1. This would require a significant raising of the WWTF berms to achieve the required hydraulic head to discharge into the Northumberland Strait;
2. Velocity in the gravity pipe would be affected by tide elevations and could lead to additional maintenance of the pipe;
3. The WWTF water levels would be directly impacted by changes to tidal elevations, resulting in inconsistent hydraulics through the facility components. This would make operation of the WWTF more challenging;

NOTES

NO.	DATE	REVISIONS	BY	APPR.
1.0	NOV 23/18	ISSUED FOR FINAL REPORT	GMG	SEB
0.0	JULY 10/18	ISSUED FOR DRAFT REPORT	DWD	SEB

GREATER SHEDIAC SEWERAGE COMMISSION
DES ÉGOUTS SHEDIAC ET BANLIEUES

crandall

PRELIMINARY ONLY
NOT TO BE USED FOR CONSTRUCTION

PROJECT TITLE

SHEDIAC EAST LONG-TERM WASTEWATER MANAGEMENT STRATEGY

SHEDIAC N.B.

DRAWING TITLE

CLIMATE CHANGE

Scale	Drawn By	Design By
10m 0 20m SCALE : 1:2000	GMG	SEB
	Checked By: SEB	Cadd Check: BW
	Sheet 1 of 1	

File Name: 3-2.DWG

Drawing No.: 3-2

NOTES

NO.	DATE	REVISIONS	BY	APPR.
1.0	NOV 23/18	ISSUED FOR FINAL REPORT	GMG	SEB
0.0	JULY 10/18	ISSUED FOR DRAFT REPORT	DWD	SEB

GREATER SHEDIAC SEWERAGE COMMISSION
DES ÉGOUTS SHEDIAC ET BANLIEUES

crandall

PRELIMINARY ONLY
NOT TO BE USED FOR CONSTRUCTION

PROJECT TITLE

SHEDIAC EAST LONG-TERM WASTEWATER MANAGEMENT STRATEGY

SHEDIAC N.B.

DRAWING TITLE

CLIMATE CHANGE

Scale	Drawn By	Design By
10m 0 20m SCALE : 1:2000	GMG	SEB
	Checked By: SEB	Cadd Check: BW
	Sheet 1 of 1	

File Name: 3-2.DWG

Drawing No. 3-2

4. A gravity outfall is exposed to the influence of sea-level rise and storm surge events, which would need to be accounted for in hydraulic design. This would result in a substantial increase in the required dyke elevation;
5. It is uncertain if the gravity could be installed through the Provincially Significant Wetland by conventional methods such as by open trench due to the wetland soil conditions. The 750-900mm diameter pipe would likely need to be installed by horizontal drilling. This size gravity pipe and type of drilling has substantial risks, challenges and costs to achieve a proper installation. Drillers for this scope of work are specialized and very limited.
6. A gravity pipe would be shallow in order to sit on the sea floor at the discharge location and could become prone to movement during freeze thaw cycles within the wetland soil conditions. There is also risk with severe ice conditions and storms along the coast.
7. It is difficult to accommodate the diffusers required for proper mixing on a gravity outfall due to headloss limitations.
8. The order of magnitude cost for a gravity outfall is estimated at three (3) times the capital cost of a forcemain drilling operation described in section 3.5.2.

3.5.2 Option 2: Pumped Outfall

The second option that was recommended in the Outfall Study included a forcemain and lift station at the end of the treatment system after the UV Building in order to pump the flows to the Northumberland Strait. This was recommended to be re-evaluated with the long-term study to confirm it would remain the option for discharging the effluent.

Preliminary Outfall/Forcemain assumptions are as follows:

- An outfall forcemain size of +/- 600 mm would be required;
- The smaller diameter requirement for the pumped option is expected to make the directional drilling operations more suitable than the large diameter piping required for the gravity option. Directional drilling is the preferred method for this type of installation due to environmental limitations as opposed to open trench installation;

- Forcemain (pressure pipe) installation is better able to accommodate the required diffusers to be installed at the end of the outfall, with the ability to add "duckbill" check valves to minimize opportunities for silt, sand, or other debris to enter the outfall;
- Ice rafting does not typically go deeper than 2 m below the lowest tide;
- The outfall will be constructed with a swab launching station at the Effluent Pumping Station end and a removable cap at the diffuser end to facilitate discharge of swabbed materials and the swabs for maintenance purposes

3.5.3 Recommendations

Although the a new more advanced treatment is being proposed that would treat Total Ammonia Nitrates (TAN), the new discharge location in the Northumberland strait would is still be required in order to achieve the required mixing. Also, the new proposed outfall location will have the added benefit of increasing the distance from the discharge point to Parlee Beach.

Based on our review of the two (2) outfall options presented above, the pumped outfall option is still the recommended approach for this facility. This recommendation is based on the factors listed in the previous sections. Furthermore, it is anticipated that the capital cost of Option (2) would be significantly lower than the gravity option. Constructability of Option (1) is also a major concern.

We believe that the benefits of Option (2) outweigh the primary drawback of constructing and operating a new pumping station.

The details and estimated costs of the proposed outfall pumping station are presented in a subsequent section.

3.6 Review of Available Treatment Technologies

When evaluating options to service the future needs of GSSC, two (2) main treatment plant types were considered. They were:

- Lagoon Type Treatment Plant
- Mechanical Type Treatment Plant

Each treatment plant type has their benefits, drawbacks and limitations. These options are described in further detail in the following sections.

3.6.1 Lagoon-Type Treatment Plant

3.6.1.1 Facultative Lagoon

Facultative (non-aerated) lagoons use natural biological processes to develop micro-organisms and algae that utilize the organic waste as a food source. They rely on natural oxygen transfer mechanisms such as photosynthesis and wind to provide sufficient oxygen to maintain an aerobic micro-organism population and avoid odours. To support the algae's photosynthetic processes this limits the depth of the cells to about 1.2 - 1.5 m to allow penetration of sunlight. To get the long retention time required for adequate treatment, area requirements are significant. Multiple cells operated in series maximize the retention time and enhance treatment efficiency. Because of the long retention time, sludge accumulation occurs slowly, so continuous sludge removal is not required. The following are key considerations with respect to facultative lagoons:

- Non-aerated lagoons are simple to operate;
- Their operation simplicity does not permit much control of the treatment process;
- Sludge production is low, and removal does not have to be done on a continuous basis; it may be required only every 15 – 20 years;
- Non-aerated lagoons have a large land area requirement;
- The "Atlantic Canada Wastewater Guidelines Manual" recommends minimum setbacks of 150 m from isolated human habitation and 300 m from built-up areas;
- Basic operator training qualifications are sufficient;

- Since the biological process rate is temperature dependent, treatment efficiency is reduced during cold weather/winter operation;
- The “Atlantic Canada Wastewater Guidelines Manual” suggests that the following effluent quality can be expected:
  - Summer to Late Fall: $CBOD_5$: 10 – 30 mg/L; TSS: 10-40 mg/L
  - Winter to Late Spring: $CBOD_5$: 25 – 70 mg/L; TSS: 20 – 60 mg/L
- These levels do not consistently meet the CCME/WSER/NBDELG Effluent Requirements;
- Non-aerated lagoons may have higher suspended solids and $CBOD_5$ levels in the effluent due to carry-through of almost neutral-density algae;
- Non-aerated lagoons offer little opportunity to achieve higher treatment standards should requirements change in the future.

Because the minimum treatment requirements cannot be met by a facultative lagoon, this treatment technology will not be further explored in subsequent sections of this Report.

3.6.1.2 Aerated Lagoon

Aerated lagoons make use of a blower system to pump air through diffusers to add the required amount of oxygen to support the biological treatment process. Because it does not rely on surface transfer of oxygen and penetration of sunlight, aerated cells can be constructed much deeper, reducing their area requirement and improving heat retention. Also, the retention time in an aerated lagoon system is much less than that of non-aerated lagoons, further reducing land area requirements. Aerated cells are followed by a cell with less aeration to act as a "polishing cell" and allow suspended organic and other material to settle. Sludge accumulation in a typical aerated lagoon occurs relatively slowly, so continuous sludge removal is not required. The following are key considerations with respect to aerated lagoons:

- Aerated lagoon systems provide the ability to control the process by being able to adjust the amount of oxygen added to match the demands of the treatment process;
- Land area requirements for aerated lagoon systems are less than for non-aerated lagoons;

- The "Atlantic Canada Wastewater Guidelines Manual" recommends aerated lagoons be located a minimum of 150 m from residences and 30 m from the nearest property line; setback area requirements are therefore considerably less than for non-aerated lagoons;
- Aerated lagoons achieve higher treatment levels than non-aerated lagoons due to a constant oxygen supply being provided and due to their greater depth retain heat for treatment even in cold-weather seasons;
- Due to its long retention time (compared to a mechanical plant), an aerated lagoon system can accept some variation in the influent flow pattern, resulting from the periodic operation of the contributing pumping stations and seasonal flow variations, without affecting the stability of the treatment process;
- The "Atlantic Canada Wastewater Guidelines Manual" suggests that the following effluent quality can be expected:
  - $CBOD_5$: 15 – 30 mg/L; TSS: 20-35 mg/L
- Sludge production is low and does not have to be done on a continuous basis; it may be required only every 15 – 20 years;
- Operator training requirements to operate the system are very similar to that of the GSSC's present treatment facility.

3.6.1.3 Additional Treatment Technologies

There are several technologies available that can be added to the lagoon process for additional treatment for other substances, including UV disinfection for pathogens, filtration to reduce phosphorus concentrations, biological treatment for ammonia reduction, and others. Several technologies selected for further review are presented in Section 3.6.3 below.

3.6.2 Mechanical-Type Treatment Facility

Mechanical treatment facilities use a series of tanks and related equipment to maintain a biological treatment process. Because of the ability to closely control the parameters of the process, retention times are relatively short. Solids removal is carried out in performance-specific settling tanks to achieve high removal levels. Sludge removal and disposal is an ongoing requirement with mechanical treatment plants. Due to the amount of mechanical equipment required (pumps, aeration systems, settling tank collection systems, sludge holding and processing equipment, etc.) and the fact that tanks are typically of reinforced concrete construction, the capital construction costs and operating and maintenance costs are usually higher. However, mechanical WWTFs provide for significant process control. The following are key considerations with respect to mechanical-type treatment facilities:

- Mechanical WWTFs have a low land area requirement;
- The "Atlantic Canada Wastewater Guidelines Manual" recommends mechanical plants be located a minimum of 150 m from residences and 30 m from the nearest property line, similar to setbacks recommended for aerated lagoons;
- Mechanical WWTFs are capable of producing a higher quality effluent without the need for additional filtration, due to the amount of process control available;
- The "Atlantic Canada Wastewater Guidelines Manual" suggests that the following effluent quality can be expected year-round: CBOD5: 10 - 25 mg/L; TSS: 10 - 25 mg/L;
- Mechanical WWTFs typically have a higher energy requirement that the previous options;
- Mechanical plants require facilities for continuous sludge removal, treatment and storage prior to disposal;
- There are more process-specific mechanical components to maintain;
- There is an increased operating cost because of the ongoing sludge removal requirement;
- Particularly for the high-rate, contact stabilization and activated sludge versions of biological treatment, process stability may be affected by the possibly irregular wastewater influent flow pattern;
- Since mechanical treatment requirements can quickly become upset during power outages and take time and attention to re-establish, stand-by power is recommended;
- Since many of the treatment parameters must be carefully controlled so as to obtain optimum treatment, the operator training requirements are typically more stringent, including a comprehensive working knowledge of the importance and relationships of the various biological growth parameters.

As part of this Study, several treatment technologies that were evaluated, as follows:

3.6.2.1 Activated Sludge

Activated sludge is the classic configuration of a mechanical plant and consists generally of aerated tank(s) and clarifier(s). The aerated tank provides treatment for the removal of organic carbon and nutrients and allows flocs (clusters of solids) to form. Aeration and agitation in the aerated tank prevents suspended material from settling out. Following the aerated tank, the suspended solids and flocs are removed in the clarifier by settling. A portion of the sludge that is formed in the clarifier is directed back into the aerated tank to provide an active population of microorganisms to support the immediate treatment of the wastewater. The surplus sludge that is generated in the clarifier must be managed and eventually disposed of off-site.

3.6.2.2 Sequencing Batch Reactor (SBR)

A variation of the activated sludge treatment plant is the SBR, which is a fill-and-draw type system where all treatment steps are performed in the same tank. The general sequence for an SBR plant includes filling the tank with raw wastewater, reaction, clarification (settling), decanting the treated wastewater, and idling. Following the decantation phase, a portion of the sludge that was generated will remain in the tank, while the remaining portion must be removed and processed.

3.6.2.3 Moving Bed Biofilm Reactor (MBBR)

The MMBR process consists of a wastewater-filled reaction tank with special plastic carriers that provide a surface where biofilm can grow. The carriers are kept in constant motion through the use of aeration and/or mechanical mixers to promote good contact with the wastewater. The biofilm that grows on the carriers provides biological treatment.

Following the MBBR tank(s), clarification is required in order to remove the suspended materials that are kept in suspension in the MBBR tank(s). No sludge wasting or re-circulating is required to sustain the MBBR tank(s); however, sludge is generated in the clarifier stage that must be managed.

3.6.3 Additional Treatment Technology

The following sections present a selection of additional treatment processes that have been reviewed as part of this Study.

3.6.3.1 Pre-treatment

Pre-treatment, also referred to as "headworks", refers to the area where preliminary treatment of the wastewater takes place. Pre-treatment is included in all concepts for the upgrade of the Cap-Brulé facility. Raw wastewater will be pumped directly to the pre-treatment equipment by a new influent lift station, which will collect all flows from the WWTF's service area. Pre- treatment is provided to prepare the wastewater for the subsequent treatment process (lagoon or mechanical plant) by removing material that could negatively impact equipment or the process. The objective is to have only organic material proceed to the treatment process. The specific components proposed for this facility are:

(a) Screening:

The screening unit removes large objects from the wastewater stream such as sticks, rags, larger floatables, rocks, etc. The screened material is mechanically collected, washed to remove organic material, compacted to reduce its volume and further dewater it, and is deposited in containers for periodic disposal to landfill. The washing, compaction and temporary storage is done in the pre-treatment building, and the wash water is returned to the influent stream for treatment. A by-pass channel is provided for the screening equipment, for maintenance purposes.

(b) Grit Removal:

It is common for inorganic grit to be picked up in the wastewater system through infiltration or inflow. This material passes through the screening unit, and it is important to remove it to prevent its accumulation in the subsequent treatment components where it tends to settle near the influent discharge area. Grit removal is accomplished in a conical chamber where the heavier grit settles to a collection hopper, while the lighter organic material stays in suspension and continues to the treatment process. The grit is pumped to equipment that settles the grit, washes it to remove organics, dewaters it and deposits it in containers for periodic disposal at the landfill. This equipment is also inside the headworks building adjacent to the grit removal equipment. The wash water is returned to the influent stream for treatment. A by-pass channel is provided for the grit tank as well, for maintenance purposes.

3.6.3.2 Submerged Attached Growth Reactor

The Submerged Attached Growth Reactor (SAGR) provides $CBOD_5$ polishing and ammonia removal in cold climates such as the GSSC facility experiences. In the SAGR, the lagoon effluent is evenly distributed into a fully aerated clean stone filled bed. As the lagoon effluent flows through the bed, the oxygen-rich environment encourages nitrifying bacterial growth on stone, which that provides the conditions required for ammonia removal (nitrification). The gravel bed is covered with a layer of peat material or mulch to prevent the unit from freezing during cold weather.

3.6.3.3 Moving Bed Biofilm Reactor

The moving bed biofilm reactor (MBBR) can be supplied as a complete mechanical treatment plant (refer to mechanical plant evaluation in Section 3.6.2), or as an add-on technology in conjunction with a lagoon system, providing additional $CBOD_5$ treatment as well as ammonia treatment. This hybrid approach uses the same technology as the mechanical plant MBBR tanks, but since most of the settling would occur in the lagoon prior to the MBBR basin, additional clarification and sludge management facilities are not typically required.

3.6.3.4 Phosphorus Treatment

Phosphorus treatment can be achieved for varying treatment objectives using several different processes. Effluent phosphorus levels of approximately 1mg/L can be achieved by adding alum between the lagoon cells, where the alum is added in a chamber to encourage good contact between the alum and the wastewater. Alum attaches with phosphorus to create floc, which is heavy enough to precipitate out of wastewater. The wastewater then passes through the subsequent lagoon cell(s) where settling occurs.

If a higher degree of treatment is desired, a filtration system can be installed at the end of the treatment system, prior to the UV system, to achieve an effluent TP concentration of roughly 0.3mg/L. This would involve the addition of a disk filter system, where a coagulant (typically alum) is added to the treated effluent, which then passes through a series of cloth filters to reduce the phosphorus concentration. Periodic backwashing is required to clean the filters.

3.6.3.5 UV Disinfection

Because of the effluent discharge to the Northumberland Strait and the nearby recreational water uses, effluent disinfection is required for this facility. An alternative of expanding the existing UV disinfection system was explored; however, due to the projected peak design flows, this would not be practical in this case. Therefore, effluent disinfection will be achieved by a new ultraviolet disinfection system, which will replace the existing system.

The UV system works by passing the effluent through the light from special UV bulbs that effectively sterilizes the bacteria, meeting the required standard of treatment. The UV bulbs are placed in protective quartz sleeves, closely spaced on a rack, and submerged in the effluent channel to allow the light to contact any bacteria. The proposed system includes automatic wiping of the bulb sleeves to maintain a high level of transmittance.

3.7 Concept Design of Long-Term Treatment Options

Several alternative wastewater treatment options were evaluated during this Study. This includes alternatives such as:

- Upgrading the existing facility within the existing lagoon footprint;
- Upgrading the existing WWTF using available GSSC-owned land;
- Enlarging the lagoon based on scenarios of "minimal land purchase" (targeting a 25-year design life);
- Full lagoon build-out utilizing all available land for expansion (targeting the 50-year loading projections);
- Review of various lagoon add-on lagoon technologies to provide additional treatment;
- Replacing the lagoon-based system with a mechanical plant; and,
- Exploring a hybrid treatment option, combining lagoon treatment with add-on mechanical technologies.

Although upgrades to the existing facility within the boundary of the current GSSC-owner property were considered, it was determined that the retention time that could be attained using the available land would not be sufficient to serve the GSSC in the long-term. Therefore, it was determined that this option did not warrant further consideration. Therefore, the following Options are presented herein:

- Option 1: Lagoon-Based Upgrades
- Option 2: Mechanical Plant
- Option 3: Hybrid Lagoon/Mechanical Plant

Both options presented also include the requirement for a new Effluent Pumping Station, as described in previous sections.

The following sections describe the proposed upgrades to the Cap-Brulé facility.

3.7.1 Effluent Pumping Station

As previously discussed, upgrades to the outfall are required regardless of the facility type or treatment technologies selected as part of the concept for this site. Furthermore, Crandall is recommending that the selected WWTF upgrades include a new effluent pumping station and forcemain outfall. Additional details on assumptions and design parameters are provided below:

- The 25-year concept peak flow rate Effluent Pumping Station is recommended to have three (3) (125-150 hp) (2+1 spare) submersible pumps rated at 490 L/s (7,767 USgpm) with 2 pumps operating during peak flow;
- Stand-by power would be recommended at this phase should regulations change and not permit the use of the existing outfall as an overflow;
- To avoid excessive on-off pumping cycles, the pumps will be controlled by a VFD (variable frequency drive) system so the pumping rate closely matches the effluent flow rate and maintains a more uniform velocity through the diffuser nozzles;
- The wet well will be kept as compact as possible because of the benefits of the VFD controlled pumping system; the wet well will include a level sensor that will send a signal to the VFD controller so that the pumping rate can be programmed to suit the effluent flow rate; the wet well will be divided into three sections so any one can be isolated for cleaning, etc., while maintaining the remaining two (2) pumps available for full pumping capacity;
- There will be a flow meter on the discharge pipe from the pumps to the outfall that will measure and totalize flow and send signals to the GSSC's WWTF SCADA system;
- The effluent pumping station could be constructed as part of the pre-treatment building to provide cost and operation efficiencies.

The pumping station has been included in the concept for the overall headworks building, as shown on Drawing 3-7 in Appendix A

3.7.2 WWTF Option 1: Lagoon Treatment System

Option 1 consists of upgrading and expanding the existing lagoon to accommodate the future loading and flows and has been broken down into two (2) phases:

- Phase 1 (25-year design life),
- Phase 2 (additional upgrades to bring the 25-year design to 50-years).

Although the current facility is meeting the WSER requirement for a “not acutely lethal” effluent, it is noted that lagoon systems alone cannot guarantee a non-lethal effluent because they are not designed to provide ammonia treatment. Because the intent of the current study is to evaluate whether a lagoon-based or mechanical plant-based treatment system would me appropriate in the long-term, all lagoon-based upgrade options have been based on including a SAGR system for ammonia removal at this stage (except for the lagoon-MBBR Hybrid option presented in Section 3.7.3, which provides ammonia treatment using MBBR technology).

In conjunction with the lagoon expansion, several components require replacement due to their current age and condition as noted in Section 2.4.1. This includes the existing aeration system and blowers, as well as the existing screw pump lift station, pre-treatment components, and service building. In addition, the existing UV building is near capacity and it is recommended that given the scale of the proposed upgrades it be replaced to accommodate the 25-year flows, with room for expansion to a 50-year design. At this stage, it is anticipated that the existing blower building could remain on-site as an additional service building once the existing blowers and other components are removed following construction.

The site piping will be re-configured based on the proposed upgrade concepts, and new flow control chambers will be installed following each pond to control the level in each cell individually. At this stage, new piping has been assumed throughout the site. However, the possibility of re-using existing piping could be evaluated during a future design phase to determine if the condition and sizing of the existing piping is sufficient.

3.7.2.1 Lagoon Treatment System – Phase 1

Section 3.1 describes the projected flows and loading for each design horizon presented herein. For the purpose of sizing a future lagoon upgrade concept, the following values were used, based on a 25-year design life:

- Average Design Flow: 12,500 cu.m./day
- Instantaneous Peak Flow Rate: 42,300 cu.m/day
- Influent BOD_5 and TSS Concentration: 148 mg/L (annual average)

(a) Proposed Upgrades

In order to accommodate the anticipated loading using a lagoon-style treatment facility, upgrades to the lagoon system will be required. The Phase 1 concept consists of the following (refer to Drawing 3-4 in Appendix A):

1. New aerated lagoon no. 1: Aerated lagoons are typically constructed with earthen dikes with slopes no steeper than 3H:1V. In order to reduce the land area requirement, a larger cell is often constructed but divided into two (2) or more cells by the installation of floating baffle curtains. These curtains have a cut-out opening at one end to allow flows to pass from one cell to another to maximize retention time. Multiple cells are advantageous because several cells operated in series provide a higher degree of treatment than a single large cell of the same volume.

   Lagoon no.1 (aerated cells #1A and #1B) will be constructed as a single HDPE-lined pond sub-divided by a floating baffle curtain. It is anticipated that a sub-drainage system will also be required to manage groundwater below the HDPE liner. These cells will have a liquid depth of 4.53 m. The proposed liquid surface elevation is +5.5 m and the top of the dikes will be at elevation +6.5 m.

   Oxygen will be supplied to each cell through the installation of a new fine-bubble aeration system, consisting of shallow-buried main air headers and floating aeration laterals. Air will be supplied by blowers as described in the WWTF Building paragraphs below.

2. New aerated lagoon no. 2 (existing polishing cell): The existing polishing cell has seen minimal change since its construction in the early 1970's. Its current shallow depth and irregular shape do not provide an efficient treatment cell in the long-term. Therefore, this cell will be re-configured as the new aerated cell no 2 by deepening the existing cell to match the bottom elevation of the existing cells (+0.97 m), expanding the cell to the south to maximize the land use, and raising the dikes to elevation +6.5 m to maximize retention time. The reconfigured cell will include the installation of a new fine-bubble aeration system, and the cell will be lined with a HDPE liner underlain by a new sub-drain system.
3. Aerated/polishing lagoon no. 3 (existing cell no. 2): The existing aerated cell no. 2 will be retained in its current physical dimensions, and the existing HDPE liner will be retained and repaired as required. The existing aeration equipment will be removed, and a new floating baffle curtain will be installed to separate the final $CBOD_5$ treatment cell from the polishing cell, which will allow material suspended in the treated wastewater to settle out before the next treatment process;
4. A new SAGR as described in Section 3.6.3.2 will be constructed within the footprint of the existing aerated cell no. 1. This will require the removal of the existing aeration equipment and HDPE liner, as well as construction of the new SAGR cells. Coarse bubble diffusers spaced evenly across the floor of the SAGR will provide oxygen to the wastewater;
5. Alum System for Phosphorus Treatment: it is anticipated that a new alum system will be installed after the first treatment cell to provide phosphorus treatment. This will include the installation of a small building to house the alum pumps and controls, as well as alum mixing chambers and associated piping.
6. New site piping, manholes and flow control chambers : new site piping as shown on the Drawings will convey the wastewater from one treatment component to another; by-pass arrangements will be included as appropriate to provide operational flexibility. A new flow control structure is anticipated at the end of each of the three (3) main cells, to control the liquid level in each pond.

The piping system also provides for WWTF protection in various "worst case" scenarios, such as loss of power to the Effluent Pumping Station. This is provided by the inclusion of an emergency treated water overflow pipe to the existing outfall channel.

7. New Headworks and Final Treatment / Disinfection Building: This building will house all of the major electrical, mechanical and control systems conveniently in a single secure location. This Building will be sized with sufficient space to accommodate the 50-year design scenario components, and will include the following:
    - *Influent screw pump station*: This station should be sized at the onset to accommodate the 50-year peak flowrate of approximately 56,000 m^3/day (650 l/s, or 10,275 USgpm). It is proposed that the station be comprised of four (4) screw pumps, each sized for half of the projected peak flow, in two (2) separate compartments to allow for each compartment to be isolated to permit future maintenance without removing the station from service. This will provide operations flexibility, where the system would normally operate based on one (1) pump running per compartment, but each compartment could handle the peak flows individually with both pumps running if required.
    - *Screening and grit removal*: screening and grit removal will be as described in Section 3.6.3.1. Due to the relatively small difference in peak flows between the 25-year scenario and the 50-year scenario, and due to the available equipment sizing, it is proposed to install equipment sized for the 50-year flows right away.
    - *Blower Room*: It is proposed that there will be two (2) blowers for the lagoon system, and one (1) dedicated blower for the SAGR (to be confirmed during preliminary design). The lagoon aeration system will normally operate with one blower, with the second unit on stand-by to ensure continuous treatment. The stand-by blower also acts as an emergency stand-by unit for the SAGR if needed.
    - *Stand-by power unit*: A diesel generator, including diesel fuel storage tank is proposed to provide emergency power to the main system components, including the influent and effluent lift stations, screens and grit removal system, blowers, UV disinfection system, and filters.

- *Phosphorus filtration*: In order to provide a higher degree of phosphorus treatment than provided by the alum system alone, a filtration system can be included in the headworks building as shown on **Drawing 3-7 in Appendix A** . This is in addition to the alum system, and is further described in Section 3.6.3.4
- *UV disinfection system*: refer to Section 3.6.3.5
- *Effluent lift station*: As noted in Section 3.5, the site conditions are not conducive to a gravity outfall. Therefore, it was recommended that an effluent lift station be installed downstream of the UV disinfection system to pump effluent to the Northumberland Strait, where sufficient mixing and dilution can occur.
- The main electrical entrance
- An office area for the operator, storage room, and washroom.

In order to accommodate the expansion of the lagoon, it is anticipated that portions of six (6) properties will need to be purchased, totalling roughly 2.6 Ha. As previously noted, the Atlantic Canada Wastewater Guidelines generally require a 150m buffer zone between a lagoon-based WWTF and the nearest residence, for new facilities. Because this is an existing facility, there are already residential properties within the buffer area. However, with the proposed expansion, current aerial mapping indicates that a total of 14 homes will fall within, or partially within the 150m buffer area.

(b) Construction Phasing

Phasing is an important aspect of any upgrades to the GSSC's Cap-Brulé treatment facility due to the need to maintain treatment levels during construction. As such, the proposed 25-year lagoon upgrade concept is based on maintaining the existing lagoon cells in operation during the first phase of construction. The anticipated upgrade sequence is as follows (to be confirmed during pre-design):

1) Construction of new aerated lagoon #1 including associated piping while maintaining existing lagoon as-is. This will also include installing the permanent by-pass piping that will be used during the next phase of construction, as well as the installation of the phosphorus treatment equipment. The existing pre-treatment, blower, and UV buildings will remain in operation during this phase;

2) At the same time as the new lagoon no. 1 is being constructed, work on the new WWTF building will begin. The new outfall to the Northumberland Strait will also be installed. By carrying out this work simultaneously with the lagoon no. 1 construction, the new pond and outfall can be used during the reconstruction of the existing cells;
3) When the new lagoon no. 1 has been constructed, all influent wastewater will be directed to it, and the contents of the existing cells would be pumped to the new lagoon no. 1 inlet chamber. When the liquid level has reached the required elevation, the aeration system diffusers and the floating baffle (to create Cells 1A and 1B) will be installed and aerated lagoon treatment will begin.
4) With the existing aeration and polishing cells now drained, work will begin on reconstructing the polishing cell to its final dimensions. The existing aeration system will also be removed and any required repairs to the HDPE liner in the existing Cell #2 made. The existing Cell #1 will be re-constructed as a new 4-cell SAGR. While these cells are out of operation, any required piping modifications will be made.
5) Once the new aerated lagoons no. 2 and no. 3 and SAGR are constructed, the flows leaving aerated lagoon no. 1 will be re-directed to them. When the cells have been filled to the required elevations, the aeration system diffusers and the floating baffle (to create aerated cell 3A and polishing cell 3B) will be installed and full aerated lagoon treatment will begin.
6) The effluent from the SAGR will be directed to the filtration and UV disinfection systems and then to the effluent lift station to be discharged to the Northumberland Strait.
7) Once the new building is complete and the existing buildings are no longer needed, they will be decommissioned / removed, or retained for storage as appropriate.

3.7.2.2 Lagoon Treatment System – Phase 2

The intent of this long-term phase is to expand on the Phase 1 upgrade to result in a concept capable of treating the 50-year projected loading. For the purpose of sizing a future lagoon upgrade concept, the following values were used, based on design life ending in 50 years:

- Average Design Flow: 23,000 cu.m./day
- Instantaneous Peak Flow Rate: 56,000 cu.m/day
- Influent BOD_5 and TSS Concentration: 160 mg/L (annual average)

(a) Proposed Upgrades

In order to accommodate the anticipated 50-year loading using a lagoon-style treatment facility, expansion of the 25-year concept will be required. The Phase 2 concept consists of the following (refer to **Drawing 3-5 in Appendix A**):

8. Aerated lagoon no. 1: Lagoon no.1 (aerated cells #1A and #1B), constructed under the first phase (25-year concept) will be re-used in this phase. Additional aeration diffusers will be required to meet the requirements of the 50-year loading.
9. New aerated lagoons no. 2 and no. 3: New aerated lagoons no 2 and no. 3 will be constructed East of the lagoon no.1 that was constructed during phase 1 to provide the additional retention time required due to the increased loading. These ponds will have a liquid depth slightly below 4.50 m to create the hydraulic grade line required for gravity flow through the WWTF, and the top of the dikes will be at elevation +6.5 m. It is anticipated that a sub-drainage system will be required to manage groundwater below the new HDPE liners.
10. Aerated/polishing lagoon no. 4 (current polishing cell): The current polishing cell that was re-constructed as the aerated lagoon no. 2 during phase 1 will be re-configured as the new aerated cell no. 4A and polishing cell no. 4B. This will involve modifications to the floating aeration system, and the installation of a new floating baffle curtain to separate the treatment cell from the polishing cell, which will allow material suspended in the treated wastewater to settle out before the next treatment process;

11. Additional SAGR Cells: four (4) additional SAGR cells will be constructed within the footprint of the existing aerated cell no. 2. This will require the removal of the then-existing floating aeration equipment and HDPE liner, as well as construction of the new SAGR cells and piping. Coarse bubble diffusers spaced evenly across the floor of the SAGR will provide oxygen to the wastewater to promote ammonia removal;
12. Alum System for Phosphorus Treatment: it is anticipated that the alum system installed during phase 1 will be retained for phosphorus treatment. Some piping modifications will be required to re-direct the flow leaving the alum mixing chambers to the new aerated lagoon no. 2.
13. New site piping, manholes and flow control chambers : similar to Phase 1, new site piping will be required as shown on the Drawings to convey the wastewater from one treatment component to another and provide by-pass arrangements for operational flexibility. A new flow control structure is anticipated at the end of each of the two (2) new main cells, as well as the reconfigured aerated/polishing cell no. 4 to control the liquid level in each pond.

    In this phase, the emergency treated water overflow pipe to the existing outfall channel will remain in-place, as well as the outfall forcemain piping.
14. New Headworks and Final Treatment / Disinfection Building: This building will be constructed in Phase 1 with sufficient space to add the 50-year scenario's components. Anticipated modifications to the building will include the following:
    - *Influent screw pump station*: This station should be sized at the onset to accommodate the 50-year peak flowrate of approximately 56,000 m^3/day (650 l/s, or 10,275 USgpm; therefore, no major modifications are anticipated.
    - *Screening and grit removal*: it is proposed to install equipment sized for the 50-year flows right away; therefore, no major modifications are anticipated.
    - *Blower Room*: It is proposed that there will be three (3) blowers for the lagoon system, and two (2) blowers for the SAGR (to be confirmed). The lagoon and SAGR aeration systems will normally operate with two blowers running each, with the third lagoon blower on stand-by to ensure continuous treatment. The stand-by blower also acts as an emergency stand-by unit for the SAGR if needed.

- *Stand-by power unit*: In the future, the diesel generator will need to be replaced with a larger unit, or only selected essential components will be on back-up power. Sizing will be considered during the preliminary design phase.
- *Phosphorus filtration*: If it is decided to proceed with the filtration system, additional banks of disk filters would be required, as indicated on **Drawing 3-7 in Appendix A.**
- *UV disinfection system*: To accommodate the 50-year flows, additional banks of lamps would be installed in the channel in the area reserved during the building's construction.
- *Effluent lift station*: Pumping equipment upgrades will be required to this station to serve the 50-year flow projections. During the Phase 1 construction, the lift station wet well and building will be sized to accommodate future pumping station upgrades.

In order to accommodate the expansion of the lagoon, it is anticipated that portions of seven (7) additional properties (additional to those purchased for Phase 1) will need to be purchased, totalling roughly 10.5 Ha. As previously noted, some residential properties within the recommended 150m buffer zone. However, with the proposed expansion, current aerial mapping indicates that a total of 20 homes will fall within, or partially within the 150m buffer area.

(b) Construction Phasing

The proposed 50-year lagoon upgrade concept is based on maintaining the then-existing lagoon cells in operation during the construction of new cells. The anticipated upgrade sequence is as follows:

1) Construction of new aerated lagoon #2 and aerated lagoon no. 3, including associated piping, while maintaining existing Phase 1 lagoon as-is.;
2) Installation of new equipment in blower and final treatment / UV rooms, including additional blowers, banks of filters and banks of UV lights;

3) When the new lagoons no. 2 and 3 have been constructed, wastewater will be directed to them following aerated lagoon no. 1, and the contents of the then-existing aerated / polishing cell no. 3 (current aerated cell no.2) would be pumped to the new lagoon no. 1 inlet chamber. When the liquid level has reached the required elevation, the aeration system diffusers will be installed and aeration diffuser layout modifications will be made in the other aerated cells. A new baffle curtain will also be installed in aerated/polishing cell no.4 to create a polishing area prior to the wastewater being directed to the new SAGR cells.
4) With the location of the four (4) additional SAGR cells now drained, work will begin on the new SAGR cells and the required piping modifications to aerated/polishing lagoon no. 4.

3.7.3 WWTF Option 2: Mechanical Treatment Facility

A review of mechanical waste water treatment plant technologies and a conceptual mechanical plant design was carried out by our sub-consultant Englobe Corp. and a summary of the selected treatment option is as follows. The complete Report can be found in Appendix C detailing the mechanical plant evaluation.

The selected process for the Cap-Brulé facility is the Moving Bed Biofilm Reactor (MBBR) technology, due to its compact footprint, proven treatment capabilities, ability to treat variable loadings, and less complex operation when compared to other mechanical plant technologies. Please see Drawing 3-6 in Appendix A for an overview of the concept.

(a) Proposed Upgrades

In general, the system consists of the following main components (following the Screening and Grit Removal), which would be the same as for the lagoon options:

1) Equalization tank in order to regulate flow to the MBBR units. Periodic sludge removal and transfer to one of the sludge transfer tanks will be required, as partial TSS settling is anticipated.

2) MBBR Tanks as described in Section 3.6.2.3. It is anticipated that two (2) parallel trains of two (2) MBBR bioreactors (four [4] MBBR tanks total) will be required, each train sized for 50% of the 50-year average flow rate. The first bioreactor in each train will primarily be for BOD_5 reduction, while the second bioreactor, downstream from the first, will provide ammonia reduction.

3) Clarification and Sludge Management: Coagulation and flocculation basins will be located downstream of the MBBR tanks before the wastewater is directed to the clarification units to remove solids and precipitated phosphorus. Clean water from the clarifier will be directed to a neutralization basin, while solids will either be skimmed off the surface (floating solids) or will be collected at the bottom of the clarifier (denser solids).

   Liquid sludge will be produced by the clarifier which must be pumped to a transfer tank, dehydrated using screw presses, and disposed of. A sludge accumulation basin is also anticipated between the sludge transfer tanks and the sludge presses. Any leachate produced by the sludge presses will be directed to the neutralization basin for pH adjustment and disinfection prior to discharge.

4) Neutralization to adjust the pH of the typically alkaline MBBR effluent to within the regulatory limits, with the addition of acid.

5) Discharge to UV Disinfection System Following the pH neutralization step, for disinfection prior to being discharged to the Northumberland Strait.

If a mechanical plant option was to be selected, a similar headworks / final treatment building would be required, as well as a mechanical plant operation building. The main components that would be included in each building are as follows:

1) Headworks / Final Treatment Building (components as described in Section 3.7.2.1):
   a. Screening and grit removal
   b. Influent screw pump station
   c. Stand-by power unit
   d. UV disinfection system
   e. Effluent lift station

f. The main electrical entrance

2) Mechanical plant operations building:
    a. Blowers
    b. Clarifiers
    c. Sludge Transfer Tanks
    d. Sludge Presses
    e. Chemical dosing systems (coagulant, polymer, anti-foam agent)
    f. An office/lab area for the operator, storage room, and washroom

Although no new land purchase is expected to be required for this option, the Atlantic Canada Wastewater Guidelines recommends a 150m buffer zone between a mechanical plant WWTF and the nearest residence, for new facilities. Current aerial mapping indicates that a total of 10 homes will fall within, or partially within the 150m buffer area of the proposed Mechanical Plant site.

(b) Construction Phasing

Since an upgrade to a mechanical plant concept would involve the construction of an entirely new treatment plant, the Option 2 upgrade concept will allow the existing lagoon in operation during the first phase of construction. The anticipated upgrade sequence is as follows:

1) Construction of new mechanical plant, including associated chambers, buildings, piping, etc. while maintaining existing lagoon as-is. This will include construction of the new outfall to the Northumberland Strait and the effluent lift station's treated water emergency overflow;
2) When the new plant has been fully constructed, all influent wastewater will be directed to it, and the system will be commissioned;
3) Once the new system is in service, the existing lagoon will be decommissioned. A decommissioning plan would be developed during a future design phase, to allow the GSSC to evaluate options for the future site use.

3.7.4 WWTF Option 3: Lagoon / MBBR Hybrid Treatment System

Similar to Option 1, Option 3 consists of upgrading and expanding the existing lagoon to accommodate the future loading and flows and has been broken down into two (2) phases:

- Phase 1 (25-year design life),
- Phase 2 (additional upgrades to bring the 25-year design to 50-years).

To meet the WSER requirement for a "not acutely lethal" effluent, the treatment process presented in Option 3 has been sized to provide ammonia removal using MBBR technology.

As was the case for Option 1, as part of this Option, several components require replacement due to their current age and condition as noted in Section 2.4.1.

The site piping will be re-configured based on the proposed upgrade concepts, and new flow control chambers will be installed following each pond to control the level in each cell individually. At this stage, new piping has been assumed throughout the site. However, the possibility of re-using existing piping could be evaluated during a future design phase to determine if the condition and sizing of the existing piping is sufficient.

3.7.4.1 Hybrid Lagoon / MBBR Treatment System – Phase 1

Section 3.1 describes the projected flows and loading for each design horizon presented herein. The following values were used for the preliminary sizing of the lagoon/MBBR concept, based on a 25-year design life:

- Average Design Flow: 12,500 cu.m./day
- Instantaneous Peak Flow Rate: 42,300 cu.m/day
- Influent BOD_5 and TSS Concentration: 148 mg/L (annual average)

(a) Proposed Upgrades

In order to accommodate the anticipated loading using a combination of lagoon-based and MBBR-based treatment, upgrades to the lagoon system will be required. The Phase 1 concept consists of the following (refer to Drawing 3-9 in Appendix A):

1. New aerated lagoon no.1 (existing polishing cell): The existing polishing cell will be re-constructed by deepening the existing cell to match the bottom elevation of the existing cells (+0.97 m), expanding the cell to the south to maximize the land use, and raising the dikes to elevation +6.5 m to maximize retention time. The reconfigured cell will be a 4.48 m deep (liquid depth), HDPE-lined aerated lagoon sub-divided by a floating baffle curtain. It is anticipated that a sub-drainage system will also be installed to manage groundwater below the HDPE liner.

   Oxygen will be supplied to each cell through the installation of a new fine-bubble aeration system, consisting of shallow-buried main air headers and floating aeration laterals. Air will be supplied by blowers as described in the WWTF Building paragraphs Headworks Building description later in this section.

2. New aerated lagoon no. 2:

   The existing aerated cell no. 2 will be retained in its current physical dimensions, and the existing HDPE liner will be retained and repaired as required. The existing aeration equipment will be removed, and a new fine-bubble floating aeration system will be installed. A floating baffle curtain is anticipated to maximize the cell's treatment.

3. Aerated/polishing lagoon no. 3 (existing cell no. 1): The existing aerated cell no. 1 will be retained, and the existing HDPE liner will remain and will be repaired as required. The existing aeration equipment will be removed, and a new floating aeration system will be installed. A new floating baffle curtain will be installed to separate the final $CBOD_5$ treatment cell from the polishing cell, which will allow material suspended in the treated wastewater to settle out before the next treatment process;

4. New MBBR Treatment Units as described in Section 3 will be constructed to the south of the existing Aerated Lagoon no.2 and East of the new Headworks and Final Treatment / Disinfection Building. This will require the construction of a new MBBR Tank with a medium bubble aeration grid to provide oxygen to the wastewater. Sieves at the tank's outlet piping ensure that the MBBR media are retained within the tanks. The wastewater will travel through the MBBR train, which consists of two (2) reactors in series: one (1) to provide final $cBOD_5$ treatment, followed by a second reactor to provide nitrification (ammonia removal)

5. Alum System for Phosphorus Treatment: it is anticipated that a new alum system will be installed after the first treatment cell to provide phosphorus treatment. This will include the installation of a small building to house the alum pumps and controls, as well as alum mixing chambers and associated piping.
6. New site piping, manholes and flow control chambers : new site piping will convey the wastewater from one treatment component to another as shown on the Drawings. To provide operational flexibility, by-pass arrangements will be included where appropriate. A new flow control structure is anticipated at the end of each of the three (3) main cells, to control the liquid level in each pond.

   The piping system also provides for WWTF protection in various "worst case" scenarios, such as loss of power to the Effluent Pumping Station. This is provided by the inclusion of an emergency treated water overflow pipe to the existing outfall channel.
7. New Headworks and Final Treatment / Disinfection Building: A building similar to that described in Section 3.7.2.1 will house all of the major electrical, mechanical and control systems. This Building will be sized with sufficient space to accommodate the 50-year design scenario components, and will include the following (components as described in Section 3.7.2.1 except as noted):
   - Influent screw pump station
   - Screening and grit removal
   - *Blower Room*: It is proposed that there will be two (2) blowers for the lagoon system. The lagoon aeration system will normally operate with one blower, with the second unit on stand-by to ensure continuous treatment. Similarly, there will be three (3) blowers for the MBBR (two [2] running, one [1] stand-by).
   - Stand-by power unit
   - Phosphorus filtration
   - UV disinfection system
   - Effluent lift station
   - The main electrical entrance
   - An office area for the operator, storage room, and washroom.

This option does not require land purchase. As previously noted, the Atlantic Canada Wastewater Guidelines generally require a 150m buffer zone between a lagoon-based WWTF and the nearest residence, for new facilities. Because this is an existing facility, there are already residential properties within the buffer area. Current aerial mapping indicates that no additional homes will fall within the 150m buffer area.

(b) Construction Phasing

The proposed 25-year lagoon upgrade concept is based on maintaining the existing lagoon cells in operation during the first phase of construction. The anticipated upgrade sequence is as follows (to be refined during preliminary design):

1) Reconstruction of existing polishing cell as new aerated lagoon no.1 including associated piping while maintaining existing aerated cells as-is. This will require draining the polishing cell in order to enlarge the cell. Also included in this step is installing the permanent by-pass piping that will be used during the next phase of construction, as well as the installation of the phosphorus treatment equipment. The existing pre-treatment, blower, and UV buildings will remain in operation during this phase;
2) At the same time as the new lagoon no. 1 (old polishing cell) is being constructed, work on the new WWTF building will begin. The new outfall to the Northumberland Strait will also be installed. By carrying out this work simultaneously with the lagoon no. 1 construction, the new pond can be used during the reconstruction of the existing aerated cells;
3) When the existing polishing cell has been constructed as new lagoon no. 1, all influent wastewater will be directed to it, and the contents of the existing aerated cells would be pumped to the new lagoon no. 1 inlet chamber. When the liquid level has reached the required elevation, the aeration system diffusers and the floating baffle (to create Cells 1A and 1B) will be installed and aerated lagoon treatment will begin.
4) With the existing aeration cells now drained, the existing aeration system will also be removed and any required repairs to the HDPE liner made. While these cells are out of operation, any required piping modifications will be made.

5) Construction of the MBBR reactors and associated site piping.
6) Completion of the new WWTF building. Commissioning of the new blowers and connection of aeration piping to the new aerated lagoons no.2 and no.3 and the MBBR.
7) Once the new aerated lagoons no. 2 and no. 3 and the MBBR have been constructed, the flows leaving the new aerated lagoon no. 1 will be re-directed to them. When the cells have been filled to the required elevations, the aeration system diffusers and the floating baffle curtains will be installed, and full treatment will begin.
8) The effluent from the MBBR will be directed to the filtration and UV disinfection systems and then to the effluent lift station to be discharged to the Northumberland Strait.
9) Once the new building is complete and the existing buildings are no longer needed, they will be decommissioned / removed, or retained for storage as appropriate.

3.7.4.2 Lagoon Treatment System – Phase 2

The intent of this long-term phase is to expand on the Phase 1 upgrade to result in a concept capable of treating the 50-year projected loading. For the purpose of sizing a long-term upgrade concept, the following values were used, based on design life ending in 50 years:

- Average Design Flow: 23,000 cu.m./day
- Instantaneous Peak Flow Rate: 56,000 cu.m/day
- Influent BOD_5 and TSS Concentration: 160 mg/L (annual average)

(a) Proposed Upgrades

In order to expand the 25-year hybrid treatment concept to 50 years, the following upgrades will be required:

1. Additional aeration diffusers in lagoon cells to accommodate increased loading;
2. Additional MBBR train identical to 25-year train (total of two (2) new reactors in series);
3. Addition of one (1) additional disc filter;
4. Modifications to New Headworks and Final Treatment / Disinfection Building: As described in Section 3.7.2.2

(b) Construction Phasing

The proposed 50-year lagoon upgrade concept is based on maintaining the then-existing lagoon cells and MBBR train in operation during the construction of new components. The anticipated upgrade sequence is as follows:

1) Installation of floating aeration equipment and new equipment in blower and final treatment / UV rooms, including additional blowers, banks of filters and banks of UV lights;
2) Construction of new MBBR Train and related site piping.

3.7.5 Budgetary Cost Estimates [Order of Magnitude]

Budgetary cost estimates (order of magnitude) have been prepared for each option to facilitate a comprehensive comparison between the presented options. Although these costs should be refined as part of preliminary design activities, the following costs have been developed based on conceptual design to compare each concept and provide the Commission with an indication of budgetary requirements. The Budgetary cost estimates for each option are summarized below:

3.7.5.1 Option 1: Lagoon Treatment Option

(a) Phase 1

The estimated preliminary construction cost for the 25-year concept (Phase 1) is +/- $30M. This includes all components including the phosphorus filtration system as shown on the drawings. The estimated total cost of the filtration system if it is decided that phosphorus treatment to this degree is not required is $1M, reducing the total estimated cost to $29M.

(b) Phase 2

Due to the uncertainties regarding flow estimates for a 50-year design period, a conceptual cost estimate was not completed at this phase. Lagoon treatment costs are largely dependent on flow volumes, which could be significantly impacted from changes to infiltration or growth patterns. While a cost estimate was not completed, one potential concept for the 50-year projections is presented in Appendix A.

A total cost for the eventual life-cycle upgrade of a lagoon option would include replacement the mechanical equipment from the Phase 1 installation, which is expected to be reaching the end of its service life when the 50-year solution.

3.7.5.2 Option 2: Mechanical Treatment Facility Option

The estimated preliminary construction cost for the mechanical plant concept is +/- $30M. This includes all components as shown on the drawings, as well as decommissioning the existing lagoon by in-filling the cells. The mechanical concept was selected to include treatment redundancy, allowing for maintenance of one (1) set of MBBR cells. As a result, while this concept is for a 25-Year design period, it has the capacity to accommodate the 50-Year estimated flows. To upgrade the conceptual mechanical plant for the 50-Year design period, it is expected that relatively minor upgrades would be required to provide redundancy in the MBBR cells.

3.7.5.3 Option 3: Lagoon/Mechanical Treatment Hybrid Facility Option

The estimated preliminary construction cost for the hybrid plant concept is +/- $30M. This includes all components as shown on the drawings, as well as all components including the phosphorus filtration system as shown on the drawings. The estimated total cost of the filtration system if it is decided that phosphorus treatment to this degree is not required is $1M, reducing the total estimated cost to $29M.

The hybrid concept was selected to include treatment redundancy, allowing for maintenance of one (1) set of MBBR cells.

A breakdown of each respective cost estimate has been provided in Appendix B for reference.

3.7.6 Comparison of Options

To thoroughly address the options of upgrading the existing lagoon versus constructing a new mechanical plant or a hybrid concept, consideration needs to be given to various factors, including the anticipated treatment level, land requirements, and operational considerations, as follows:

1) **Anticipated Treatment Level:** As indicated in the following table, each option provides a similar level of treatment.
2) **Land Purchase Requirements:** Aerated lagoons require more land area than mechanical plants. The Phase 1 lagoon concept required a land purchase or approximately 2.6 Ha, while the Phase 2 upgrade would require the purchase of an additional 10.5 Ha. The mechanical plant option and the hybrid lagoon/MBBR option would not require that additional land be purchased.
3) **Operational Stability:** Although both lagoon treatment and mechanical treatment processes are considered to be stable, aerated lagoons generally provide greater process stability than mechanical plants due to their greater retention time, which acts as a buffer to variations in flow rate or wastewater strength. The hybrid lagoon/MBBR option could be expected to be somewhat more stable than a mechanical plant, but slightly less stable than the lagoon-only option due to its reduced retention times.
4) **Operator Training:** Mechanical plants typically require a higher degree of operator training due to the additional components to monitor, and the requirement for sludge handling and disposal.
5) **Operational Complexity:** In general, an aerated lagoon system is simpler to operate, requiring primarily only matching of aeration system operation with required dissolved oxygen levels. The addition of an alum system for phosphorus treatment adds a small degree of complexity. By comparison, the MBBR process requires matching aeration as well as managing sludge continuously, and the use of multiple chemicals.

   The Hybrid option falls in between Option 1 and Option 2 in terms of operational complexity, requiring more operation than a lagoon, but slightly less than the Mechanical Plant MBBR. The hybrid option does not require daily sludge management as sludge is returned to the lagoons for storage until removal is required (roughly every 2-3 years).

6) **Sludge Handling:** A lagoon is typically expected to require sludge removal every +/- 15 years, on average. Since lagoon sludge removal is relatively infrequent, it is typically carried out by a contractor hired to clean the lagoon. However, a mechanical plant produces sludge continually, which must be managed by the operator by stockpiling, de-watering, and eventually disposing of it. The hybrid option required sludge removal every 2-3 years typically, and options for sludge removal should be considered during design.
7) **Chemical Requirements:** In mechanical treatment plants, chemicals may be used to assist various processes. Chemicals may be used for coagulation and flocculation, for sludge conditioning and dewatering, and for pH adjustment. For the lagoon and the hybrid concepts, the use of chemicals is only required for phosphorus treatment.

Several of these factors impact the operating and maintenance costs of the facilities, such as operator time requirements, sludge handling and disposal, and chemical requirements. Others are not entirely cost-based considerations but are important to be aware of when making a comparative decision. The following table summarizes the key considerations of each option:

Table 3-7: Comparison of WWTF Options

CRITERIA	OPTION 1 (LAGOON + SAGR) PHASE 1	PHASE 2	OPTION 2 (MBBR MECH. PLANT)	OPTION 3 (HYBRID LAGOON/MBBR)
Anticipated Treatment Level				
$CBOD_5$ (mg/L)	15	15	20	15
TSS (mg/L)	20	20	20	15
TAN (mg/L)	1/5*	1/5*	1	2/5*
TP (mg/L)	1/0.3**	1/0.3**	0.5	0.5
E.coli (MPN/100mL)	200	200	200	200
Land Purchase requirements (Ha)	2.6	10.5	0	0
Operational Stability	Very stable		Stable	Stable
Operation & Maintenance Requirements				
Operator Training	Normal		More Advanced	Moderately Advanced
Operational Complexity	Simple		More Complex	Moderately Complex
Sludge Handling	Removal (±15 years)		Continuous	Removal (2-3 yrs)
Chemical Use	One (Alum)		Several (anti-foam, coagulant, polymer, acid)	One (Coagulant)
Capital Cost	$30M***		$30M	$30M***

* summer / winter

** without filtration system (addition of alum between lagoons only) / with filtration system

*** Cost for Phase 1.

3.7.7 Selected Option for Preliminary Design

Based on the results of a comparison between a Lagoon-type and a Mechanical-type treatment facility, as summarized in Table 3-5, it is proposed that a Lagoon-type treatment facility with MBBR enhancement (Option 3) be advanced into preliminary design. This was selected as the preferred option as it is capable of providing a comparable level of treatment to a mechanical plant while requiring significantly less operator input and training. It is anticipated that the additional monitoring requirements for a mechanical plant would necessitate the addition of one or more operators to the Commission. Furthermore, sludge management requirements are minimal with Option 3, as the lagoon cells act as clarifiers. Capital and operational costs are also estimated to be significantly less with this option. Additional considerations to be explored during preliminary design are listed below:

1. It is recommended that the project team evaluate the merit of designing any facility components for the 50-year design flow projections presented herein. It is proposed that due to the uncertainty of these projections, and their impact on the overall scale and cost of the required upgrades, that preliminary design proceed for the 25-year design flow projection.
2. Due to the magnitude of the recommended upgrades, it is proposed that a detailed review of phasing options be completed during preliminary design activities. It is likely that the Commission will be able to partition this project into phases that meet the current needs of the WWTF in the short term while positioning themselves to meet the full 25-year concept in the medium term.

It is proposed that this type of modified lagoon facility would serve the Commission well in both the short and long term.

3.8 Energy Efficiency and Renewable Energy

3.8.1 Renewable Energy

Renewable energy technologies have recently been more popular in wastewater treatment plants and can provide long term energy solutions for treatment processes and pumping operations. Energy costs associated with operating a WWTF facility typically account for 15-40% of a facilities operation. Conventional energy cost fluctuates, and this cost is directly passed down to the end users. Renewable energy sources, such as solar, wind and heat recovery have become the more attractive recourses for WWTF to reduce and stabilize these costs.

3.8.1.1 Wind Power

Wind energy can be used to either pump water mechanically (using windmills), or produce electricity from a wind turbine to pump, treat and disinfect water. Windmills or turbines could be places through the WWTF or if permitted within the Northumberland Strait for better conditions for generating power. The technology however, would have a substantial capital cost and continued maintenance is vital for continued efficiency.

3.8.1.2 Solar Power

Similar to wind power, solar power can be used particularly in treating wastewater to offset energy costs by collecting the sun energy using a range of ever evolving technologies. That being said, this option requires a significant amount of real estate in order to produce the energy required to offset conventional energy sources.

3.8.1.3 Heat Recovery

The basic principle with heat recovery technology in a WWTF application is to captures the heat off the effluent to use where the heat is needed such as inside building or to heat the domestic water. This process can remain efficient as long as the effluent maintains a temperature above 11°C. Using this heat recovered in building and domestic water can 1) reduce spikes in heating costs and 2) does not affect wastewater-treatment operations or processes.

These options are very detailed in nature and would be evaluated during preliminary and detailed design in order to determine the best application for the treatment process chosen.

3.8.2 Energy Efficiency

As previously stated, energy costs can represent a significant portion of the overall operating costs for a wastewater treatment facility. Therefore, it is recommended that various options be evaluated during preliminary and detailed design of the proposed WWTF upgrades to reduce and optimize the overall energy expenditure long term. Investment in energy efficient technology and considering energy usage in design activities can result in substantial savings. The design efforts should consider the following elements as a minimum.

3.8.2.1 Pumping Energy

Energy required for pumping stations can be a considerable cost in the overall cost to operate a WWTF. In order to minimize pumping costs, various technologies have been evaluated such as screw pumps, submersible pumps, above-ground pumps, and propeller pumps. Based on our initial review, screw pumps appear to be an efficient option due to the relatively low head and large flow variations. These pumps are very effective as they can pump at different rates of flow depending on how high the water is in the wet well, more inlet flow more pumping capacity with no change in screw speed.

For the effluent pumping station, due to the required forcemain length and overall head requirement, submersible pumps with variable frequency drives (VFDs) are the preferred approach.

A more detailed analysis of pump type selection, configuration and operating parameters should be completed during preliminary design.

3.8.2.2 Aeration Energy

The floating fine-bubble diffuser technology proposed for this application provides an efficient means of transferring oxygen to the lagoon system. Each individual diffuser is suspended from the floating air lateral to its position near the bottom of the lagoon cell and consists of multiple membranes extending away from the diffuser's centre. This results in an evenly distributed bubble pattern, resulting in a high oxygen transfer rate with minimal losses.

To further improve the system efficiency, the blowers are typically controlled by variable frequency drives, which allows the actual air produced to be matched more accurately to the required dissolved oxygen levels in the lagoon, reducing the power consumption during periods of low oxygen demand.

4 Conclusions and Recommendations

The following section summarizes the conclusions and recommendations presented in the preceding report:

1. In support of preliminary and detailed design activities it is recommended that the Commission complete flow monitoring at both the influent and effluent ends of the existing WWTF. This data would be used to validate the long-term data available through SCADA and would allow the design team to refine flow and loading estimates for future upgrades.

2. The Commission should continue efforts related to I/I identification and reduction. Review of flow metering data and comparisons with theoretical flow estimates indicate that I/I continues to have a significant influence on the overall flows at the WWTF. Benefits of reducing I/I include reduced pumping costs, more stable plant operation, reduced overflows, and extended capacity life for hydraulic components.

3. The existing facility, while continuing to produce effluent results consistent with the Certificate of Approval to Operate (CAO), is approaching its design capacity. This is consistent with the design life of the upgrades completed at this facility in 1994. Therefore, improvements will likely be required in the short term to continue to meet the CAO objectives.

4. Hydraulic modelling of the existing facility suggests a hydraulic restriction between Cell No.2 and Cell No.3 during high flow events. Furthermore, results from the hydraulic model suggest that adjustments to the UV bypass chamber could be beneficial at the UV building to allow flows to be managed during high flow events. The bypass chamber is currently limiting the capacity of the UV system. If these components are not replaced in the short term as part of an overall upgrade to the WWTF, additional investigation should be completed at these areas to provide a temporary solution to the noted hydraulic capacity concerns.

5. Through a review of the major components at the existing WWTF, it is evident that many of the components installed during the last life-cycle upgrade (1994) are reaching the end of their service life and will require attention in the short term (0-5 years).

6. As one of the results of the recently completed Environmental Risk Assessment (ERA), the outfall is currently not meeting the CCME requirements for mixing levels. As recommended in the subsequent report entitled *Feasibility Study: Cap-Brulé WWTP Outfall (2015)* a new outfall location approximately 350m off-shore is required to achieve the required mixing ratios for the facility. This upgrade should be completed in the short term (0-5 years), whether part of an overall upgrade of the WWTF or independently, to remain in compliance with CCME requirements. If completed independently, it is recommended that the required infrastructure be located according to the concepts presented herein for the overall WWTF upgrade.

7. Flow and loading projections were completed using a population growth rate of 2% per year (based on historical growth rates in Shediac) for a 25-year and 50-year planning period. Average flows are anticipated to increase by approximately 46% in 25 years and 136% in 50 years. Peak flows are expected to increase for the two scenarios by 27% and 68% respectively.

8. A cursory review of the WWTF siting was completed to comment on whether the facility's location is still ideal, considering recent growth patterns, sensitive environments, the requirement for a new outfall, climate change, etc. It is proposed that the costs associated with relocating the facility are not justified by any potential benefits. Therefore, it is recommended that the facility remain in its current location.

9. The following treatment objectives are recommended for the upgraded facility, based on a review of future regulations and best practice sites:

   - $CBOD_5$: 25 mg/L
   - TSS: 25 mg/L
   - Un-ionized Ammonia: 1.25 mg/L

- TAN: 5.0 mg/L
- TP: 1.0 mg/L
- E.coli: 200 MPN / 100 mL

10. The impacts of relative sea-level rise (associated with climate change) were reviewed for both the existing facility and proposed upgrades. In general terms, sea-level rise projections suggest that the site does not need to be raised, but that the hydraulics of the existing and proposed outfalls could be impacted.

11. Options for the new required outfall, as previously presented in Feasibility Study: Cap Brulé WWTP Outfall (2015), were re-visited in light of the overall concept for site upgrades. It is recommended that the Commission consider a pumped outfall as the preferred solution. The reasons for this recommendation include:

    a) Constructability of a pressure pipe option is better than a larger gravity pipe
    b) Difficulties in accommodating the required diffusers at the end of the outfall due to headloss limitations with a gravity option.
    c) Water level in the facility is directly influenced by sea-levels in the gravity option and is therefore sensitive to the impacts of climate change.
    d) Concerns with maintenance of a gravity option due to lower velocities through the larger required pipe.

12. Available treatment technologies were reviewed in detail for the required WWTF upgrades. Primarily, a comparison was made between a lagoon-type facility, a mechanical-type facility and a hybrid Lagoon/MBBR facility. Through an evaluation of the comparative costs and benefits of each facility type, it is recommended that the Commission proceed to preliminary design with a Hybrid option. The following additional recommendations are presented:

    a) It is recommended that the project team evaluate the merit of designing any facility components for the 50-year design flow projections presented herein. It is proposed that due to the uncertainty of these projections, and their impact on the overall scale and cost of the required upgrades, that preliminary design proceed for the 25-year design flow projection.

b) Due to the magnitude of the recommended upgrades, it is proposed that a detailed review of phasing options be completed during preliminary design activities. It is likely that the Commission will be able to partition this project into phases that meet the current needs of the WWTF in the short term while positioning themselves to meet the full 25-year concept in the medium term.

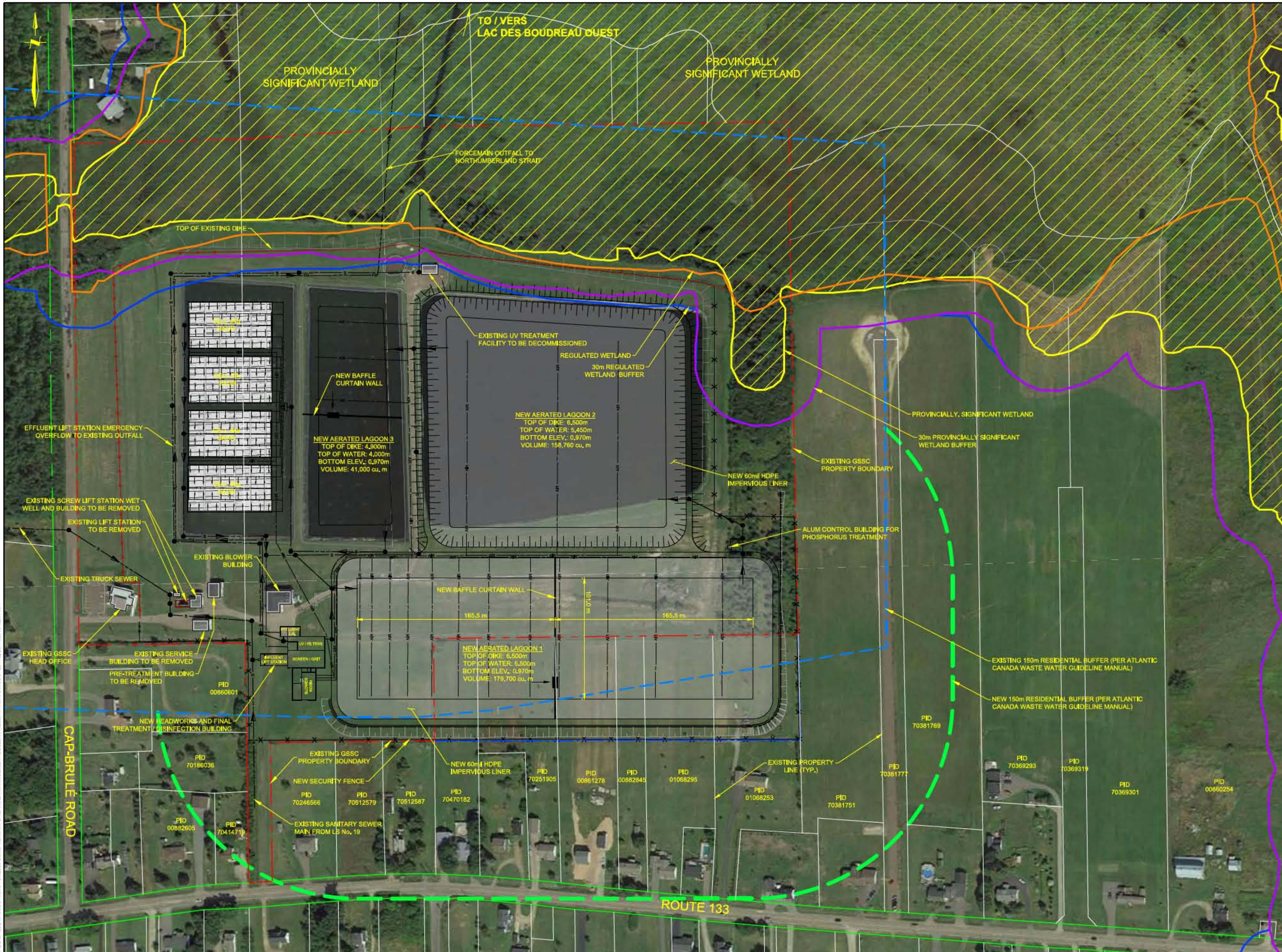
13. Order of magnitude cost estimates were established to assist in comparing each option. These estimates include a contingency (20%), an allowance for engineering (15%) and allowances for environmental and geotechnical studies. The estimated costs are summarized below:

    a) Lagoon Type Facility
        - Phase 1: 25-year Concept: $30M
    b) Mechanical Type Facility
        - Phase 1: 25-year Concept $30M
    c) Lagoon/Mechanical Hybrid Concept
        - Phase 1: 25-year Concept $30M

14. It is recommended that the Commission proceed to preliminary design immediately following selection of the preferred conceptual option. Due to the nature of the required upgrades, there are considerable investigation, permitting and design activities that are required prior to commencing construction of the WWTF improvements. Furthermore, completing preliminary design would allow the Commission to be positioned to request funding through the next round of the Building Canada Fund (BCF), which is anticipated to open for applications in the Fall of this year (2018).

Appendix A:

Treatment Plant Concepts



KEY PLAN

N.T.S.

NORTHUMBERLAND STRAIT
CAP BIMET
LAC DES BOUDREAU
TO / VERS SHEDIAC
ROUTE 133
TO / VERS CAP-PELÉ

LEGEND

——S——	NEW GRAVITY SEWER
——AIR——	NEW AIR LINE
——FM——	NEW FORCEMAIN
	PROVINCIALLY SIGNIFICANT WETLAND
	PROVINCIALLY SIGNIFICANT WETLAND (30m BUFFER)
	REGULATED WETLAND
	REGULATED WETLAND (30m BUFFER)

NO.	DATE	REVISIONS	BY	APPR.
1.0	NOV 23/18	ISSUED FOR FINAL REPORT	GMG	SEB
0.0	JUL 10/18	ISSUED FOR DRAFT REPORT	DWD	SEB

GREATER SHEDIAC SEWERAGE COMMISSION
DES ÉGOUTS SHEDIAC ET BANLIEUES

crandall

PRELIMINARY ONLY
NOT TO BE USED FOR CONSTRUCTION

PROJECT TITLE

SHEDIAC EAST LONG-TERM WASTEWATER MANAGEMENT STRATEGY

SHEDIAC N.B.

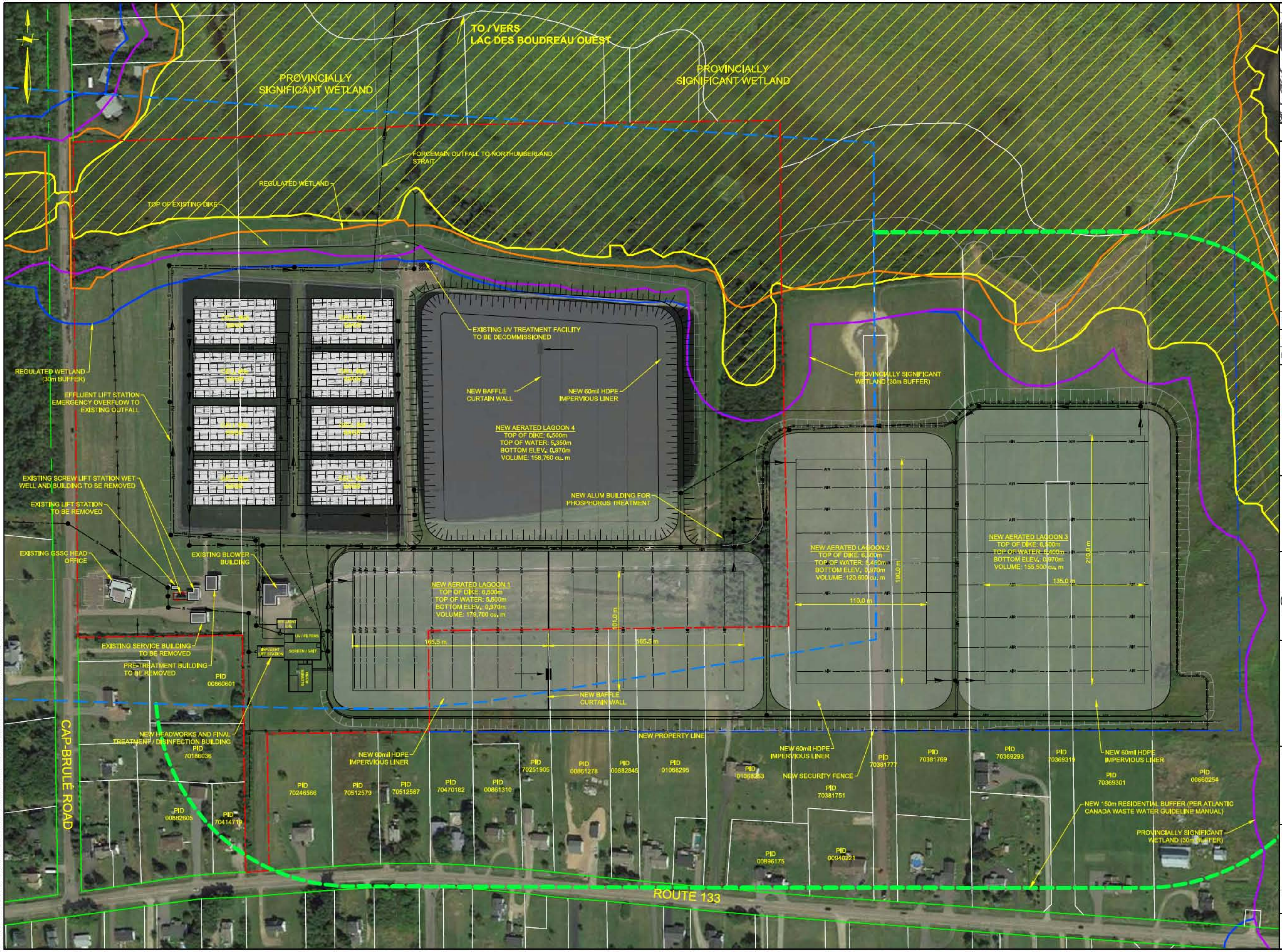
DRAWING TITLE

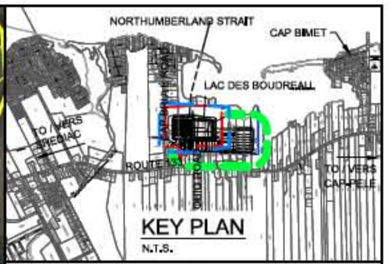
NEW AERATED LAGOON
50 YEAR CONCEPT
(PHASE 1 - 25 YEARS)

Scale	Drawn By	Design By
15m 0 30m SCALE : 1:3000	GMG	SEB
	Checked By PP	Cadd Check BW
	Sheet 1 of 1	

File Name 3-4.DWG

Drawing No. 3-4





LEGEND

—S—	NEW GRAVITY SEWER
—AIR—	NEW AIR LINE
—FM—	NEW FORCEMAIN
	PROVINCIALLY SIGNIFICANT WETLAND
	PROVINCIALLY SIGNIFICANT WETLAND (30m BUFFER)
	REGULATED WETLAND
	REGULATED WETLAND (30m BUFFER)

NO.	DATE	REVISIONS	BY	APPR.
1,0	NOV 23/18	ISSUED FOR FINAL REPORT	GMG	SEB
0,0	JULY 10/18	ISSUED FOR DRAFT REPORT	DWD	SEB

GREATER SHEDIAC SEWERAGE COMMISSION
DES ÉGOUTS SHEDIAC ET BANLIEUES

crandall

PRELIMINARY ONLY
NOT TO BE USED FOR CONSTRUCTION

PROJECT TITLE

SHEDIAC EAST LONG-TERM WASTEWATER MANAGEMENT STRATEGY

SHEDIAC N.B.

DRAWING TITLE

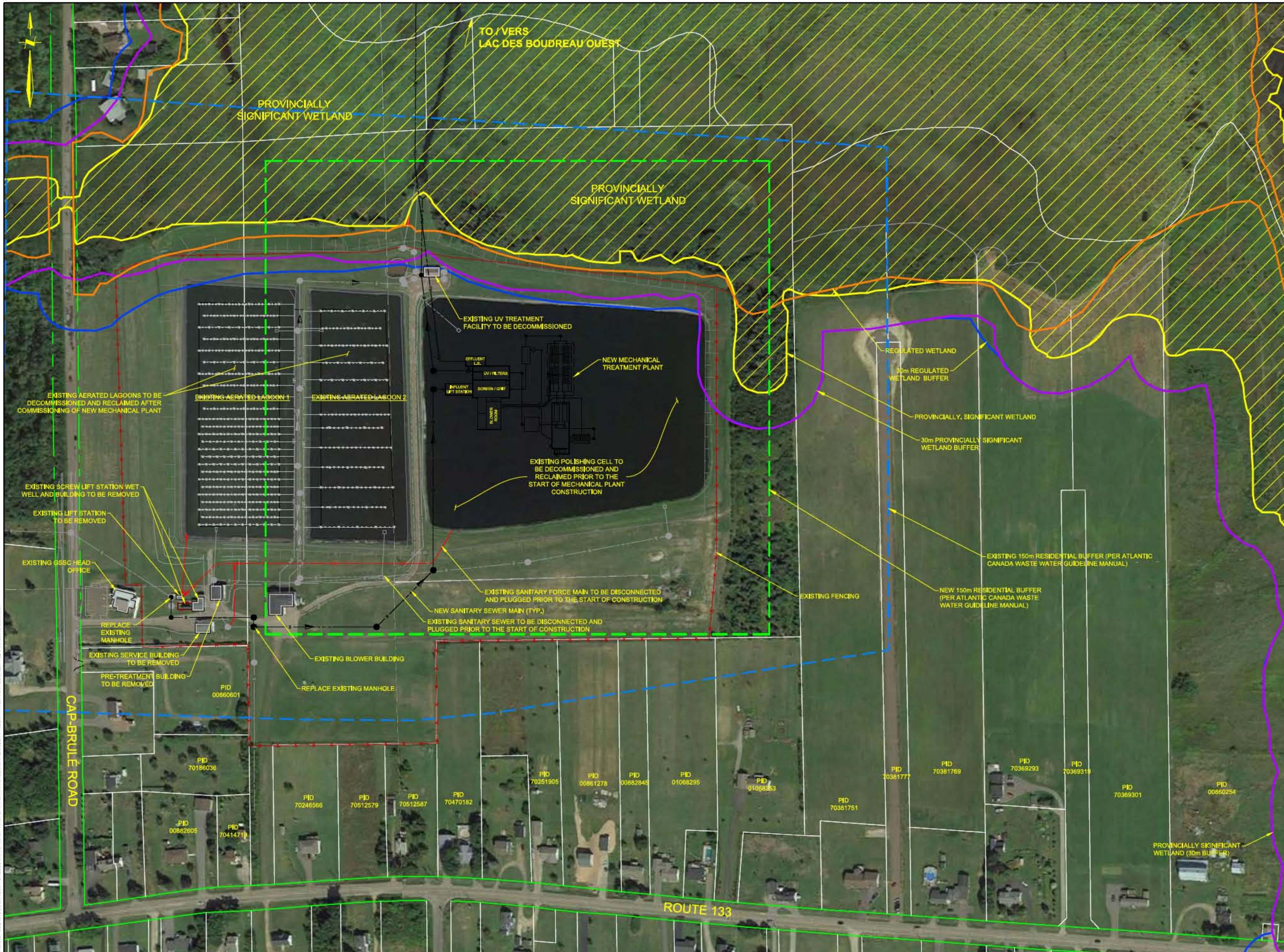
NEW AERATED LAGOON 50 YEAR CONCEPT (PHASE 2)

Scale	Drawn By	Design By
SCALE : 1:3000	AC	LEL
	Checked By: LEL	Cadd Check: BW
	Sheet 1 of 1	

File Name: 3-5.DWG

Drawing No. 3-5

G:\CADD\SHEDIAC\GSSC\17250 - SHEDIAC LONG-TERM WASTEWATER MANAGEMENT\DESIGN\PRESENTATION DWGS\3-5.DWG, 23/11/2018 4:29 PM



LEGEND

NEW GRAVITY SEWER

NEW AIR LINE

NEW FORCEMAIN

PROVINCIALLY SIGNIFICANT WETLAND

PROVINCIALLY SIGNIFICANT WETLAND (30m BUFFER)

REGULATED WETLAND

REGULATED WETLAND (30m BUFFER)

NO.	DATE	REVISIONS	BY	APPR.
1.0	NOV 23/18	ISSUED FOR FINAL REPORT	GMG	SEB
0.0	JULY 10/18	ISSUED FOR DRAFT REPORT	DWD	SEB

GREATER SHEDIAC SEWERAGE COMMISSION
DES ÉGOUTS SHEDIAC ET BANLIEUES

crandall

PRELIMINARY ONLY
NOT TO BE USED FOR CONSTRUCTION

PROJECT TITLE

SHEDIAC EAST LONG-TERM WASTEWATER MANAGEMENT STRATEGY

SHEDIAC N.B.

DRAWING TITLE

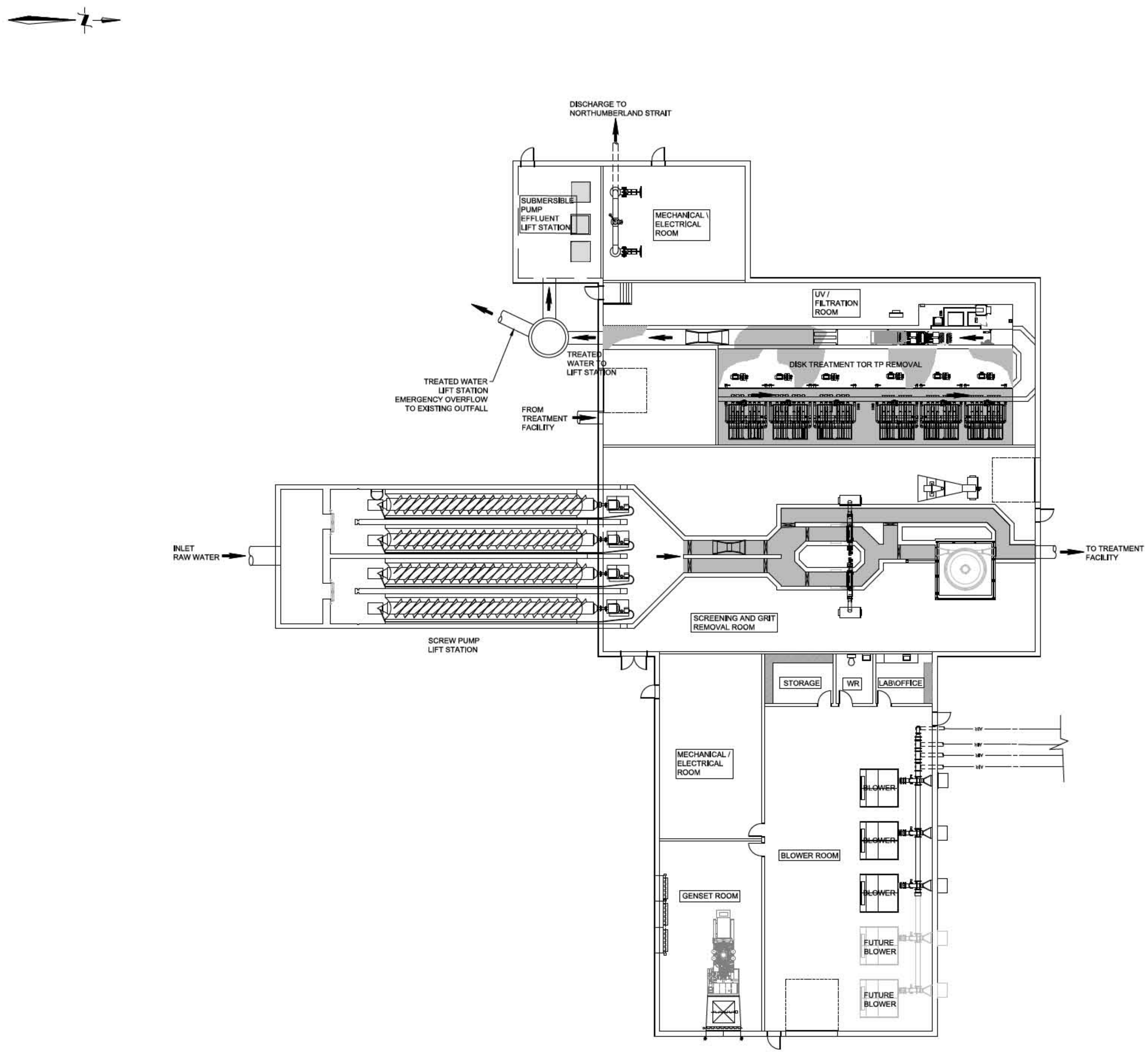
OVERALL SITE PLAN - MECHANICAL PLANT

Scale	Drawn By	Design By
15m 0 30m SCALE : 1:3000	AC	LEL
	Checked By LEL	Cadd Check BW
	Sheet 1 of 1	

File Name 3-6.DWG

Drawing No. 3-6

G:\CADD\SHEDIAC\GSSC-17250 - SHEDIAC LONG-TERM WASTEWATER MANAGEMENT\DESIGN\PRESENTATION DWGS\3-6.DWG, 11/07/2018 8:58 AM



NOTES

NO.	DATE	REVISIONS	BY	APPR.
1.0	NOV 23/18	ISSUED FOR FINAL REPORT	GMG	SEB
0.0	JULY 10/18	ISSUED FOR DRAFT REPORT	DWD	SEB

GREATER SHEDIAC SEWERAGE COMMISSION DES ÉGOUTS SHEDIAC ET BANLIEUES

crandall

PRELIMINARY ONLY
NOT TO BE USED FOR CONSTRUCTION

PROJECT TITLE

SHEDIAC EAST LONG-TERM WASTEWATER MANAGEMENT STRATEGY

SHEDIAC N.B.

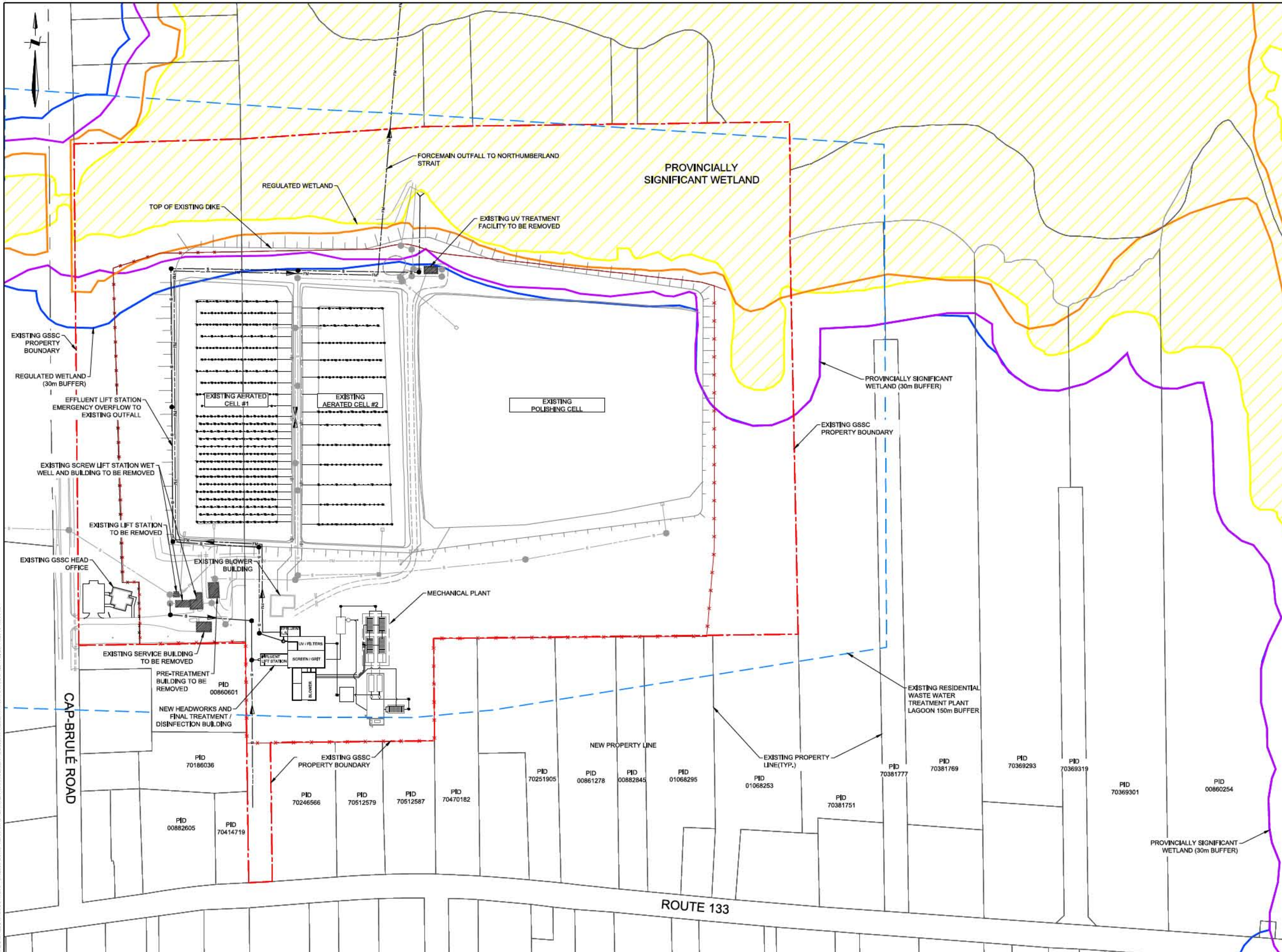
DRAWING TITLE

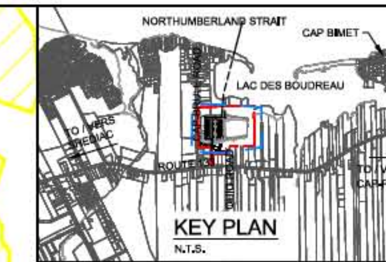
PRE-TREATMENT BUILDING

Scale	Drawn By	Design By
1.5m 0 3m SCALE : 1:300	DWD	PP
	Checked By LEL	Cadd Check BW
	Sheet 11 of 11	

File Name 3-7.DWG

Drawing No. 3-7





LEGEND

—S—	NEW GRAVITY SEWER
—AIR—	NEW AIR LINE
—FM—	NEW FORCEMAIN
	PROVINCIALLY SIGNIFICANT WETLAND
	PROVINCIALLY SIGNIFICANT WETLAND (30m BUFFER)
	REGULATED WETLAND
	REGULATED WETLAND (30m BUFFER)

NO.	DATE	REVISIONS	BY	APPR.
1.0	NOV 23/18	ISSUED FOR FINAL REPORT	GMG	SEB
0.0	JULY 10/18	ISSUED FOR DRAFT REPORT	DWD	SEB

GREATER SHEDIAC SEWERAGE COMMISSION DES ÉGOUTS SHEDIAC ET BANLIEUES

crandall

PRELIMINARY ONLY
NOT TO BE USED FOR CONSTRUCTION

PROJECT TITLE

SHEDIAC EAST LONG-TERM WASTEWATER MANAGEMENT STRATEGY

SHEDIAC N.B.

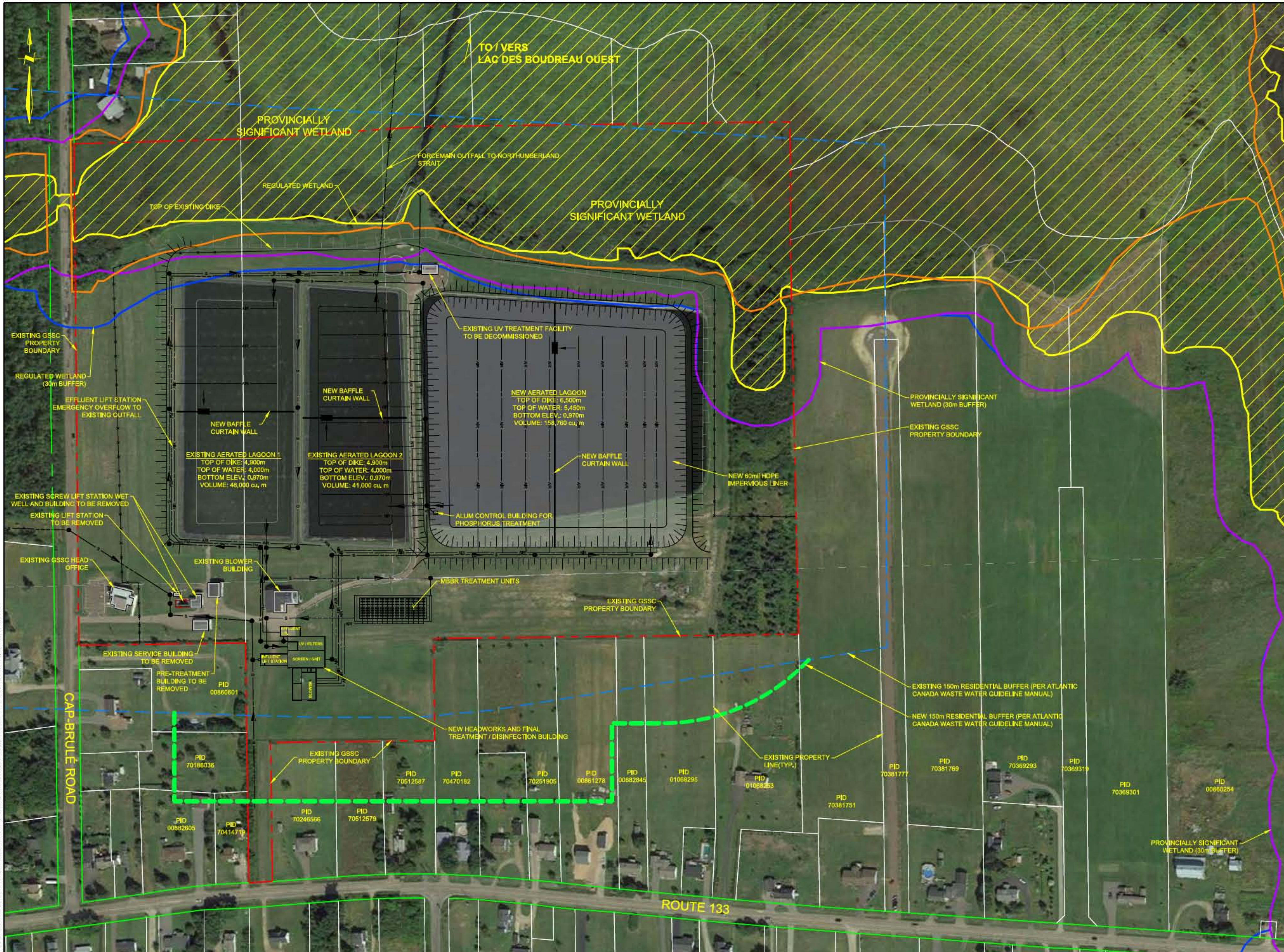
DRAWING TITLE

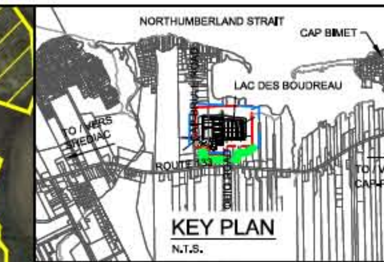
PLAN VIEW MECHANICAL WWTP OPTION

Scale	Drawn By	Design By
15m 0 30m SCALE : 1:3000	GMG	SEB
	Checked By SEB	Cadd Check BW
	Sheet 1 of 1	

File Name 3-8.DWG

Drawing No. 3-8





KEY PLAN
N.T.S.

LEGEND

——S——	NEW GRAVITY SEWER
——AIR——	NEW AIR LINE
——FM——	NEW FORCEMAIN
(yellow hatch)	PROVINCIALLY SIGNIFICANT WETLAND
(purple line)	PROVINCIALLY SIGNIFICANT WETLAND (30m BUFFER)
(orange line)	REGULATED WETLAND
(blue line)	REGULATED WETLAND (30m BUFFER)

NO.	DATE	REVISIONS	BY	APPR.
1.0	NOV 23/18	ISSUED FOR FINAL REPORT	GMG	SEB
0.0	JULY 10/18	ISSUED FOR DRAFT REPORT	DWD	SEB

GREATER SHEDIAC SEWERAGE COMMISSION
DES ÉGOUTS SHEDIAC ET BANLIEUES

crandall

PRELIMINARY ONLY
NOT TO BE USED FOR CONSTRUCTION

PROJECT TITLE

SHEDIAC EAST LONG-TERM WASTEWATER MANAGEMENT STRATEGY

SHEDIAC N.B.

DRAWING TITLE

NEW AERATED LAGOON WITH MBBR TREATMENT 25 YEAR CONCEPT

Scale	Drawn By	Design By
15m 0 30m SCALE : 1:3000	GMG	SEB
	Checked By PP	Cadd Check BW
	Sheet 1 of 1	

File Name 3-9.DWG

Drawing No. 3-9

D:\CADD\SHEDIAC\GSSC17250 - SHEDIAC LONG-TERM WASTEWATER MANAGEMENT\DESIGN\PRESENTATION DWGS\3-9.DWG, 23/11/2018 4:35 PM

Appendix B:

Cost Estimates (Order of Magnitude)

Greater Shediac Sewerage Commission - Long Term Study
Shediac, NB

25 YEAR LAGOON CONCEPT & NEW WASTE WATER TREATEMENT PLANT
(PHASE 1 LAGOON OPTION)

CONCEPTUAL COST ESTIMATE - JULY 1, 2018
Project No. 17250

Item No.	Description	Unit	Estimated Quantity	Unit Price	Total Cost
1.	*Construction Facilities*	Lump Sum	1		$ 27,600
2.	*Dust Control (calcium chloride)*	Lump Sum	1		$ 5,000
3.	*Removals*	Lump Sum	1		$ 355,000
4.	*Earthworks*	Lump Sum	1		$ 1,089,750
5.	*Sanitary Piping*	Lump Sum	1		$ 4,232,500
6.	*WWTP Accessories (Boat and Motor)*	Lump Sum	1		$ 8,500
7.	*WWTP Air Piping System*	Lump Sum	1		$ 1,309,500
8.	*Security Fencing*	Lump Sum	1		$ 82,000
9.	*Lagoon No. 1 Construction*	Lump Sum	1		$ 1,799,100
10.	*Lagoon No. 2 Construction*	Lump Sum	1		$ 1,987,975
11.	*Lagoon No. 3 Construction*	Lump Sum	1		$ 202,400
12.	*SAGR Construction*	Lump Sum	1		$ 3,293,000
13.	*Headworks, UV, & Efluent Pumping Building Construction*	Lump Sum	1		$ 9,507,700
				Sub-Total :	$ 23,900,025
				Contingency Allowance :	$ 2,000,000
				Engineering Allowance (+-12%) :	$ 3,000,000
				Environmental Study Allowance :	$ 100,000
				Geotechnical Allowance :	$ 50,000
				Sub Total :	$ 29,050,025
				15% HST :	$ 4,357,504
				HST Rebate :	$ (3,112,503)
				GRAND TOTAL (HST Incl.) :	**$ 30,295,026**

* Not Including Land Purchase

Greater Shediac Sewerage Commission - Long Term Study
Shediac, NB

MECHANICAL TREATMENT PROCESS & NEW WASTE WATER TREATEMENT PLANT
CONCEPTUAL COST ESTIMATE - JULY 18, 2018
Project No. 17250

Item No.	Description	Unit	Estimated Quantity	Unit Price	Total Cost
1.	*New Equilization Tank, Complete*	lump sum	1		$ 478,150
2.	*Bioreactor Foundations incl. Coagulation & Flocculation Basins, Complete*	lump sum	1		$ 1,121,000
3	*New Sludge Processing Building, Structural & Architectural, Complete*	lump sum	1		$ 683,770
4.	*Sludge Accumulation Basin, Complete*	lump sum	1		$ 218,350
5.	*PH Neutralization Pond, Complete*	lump sum	1		$ 358,150
6.	*Burried Air Distrbution Piping incl Fittings, Complete*	lump sum	1		$ 104,685
7.	*Sanitary Sewerage Piping, Complete*	lump sum	1		$ 3,512,175
8.	*Sludge Processing Building*	lump sum	1		$ 1,000,000
9.	*Headworks, UV, & Efluent Pumping Building Construction*	lump sum	1		$ 8,227,700
10.	*MBBR Mechanical Equipment*	lump sum	1		$ 5,000,000
11.	*Removals*	lump sum	1		$ 355,000
12.	*Decommissioning of Existing Lagoons*	lump sum	1		$ 1,825,000
				Sub-Total :	$ 22,883,980
				Contingency Allowance :	$ 2,750,000
				Engineering Allowance :	$ 3,000,000
				Environmental Permit Allowance :	$ 100,000
				Geotechnical Allowance :	$ 50,000
				Sub Total :	$ 28,783,980
				15% HST :	$ 4,317,597
				HST Rebate :	$ (3,083,998)
				GRAND TOTAL (HST Incl.) :	**$ 30,017,579**

*** Not Including Land Purchase**

Greater Shediac Sewerage Commission - Long Term Study
Shediac, NB

GREATER SHEDIAC SEWERAGE COMMISSION DES ÉGOUTS SHEDIAC ET BANLIEUES

25 YEAR LAGOON CONCEPT & NEW WASTE WATER TREATEMENT PLANT
w/ MBBR UNIT
CONCEPTUAL COST ESTIMATE - October 9, 2018
Project No. 17250

Item No.	Description	Unit	Estimated Quantity	Unit Price	Total Cost	Phase 1 (New Aerated Cell, New Aeration & New Headworks Building)	Phase 2 (New Sanitary Forcemain, Effluent Pumps and UV)	Phase 3 (New MBBR and Disc Filters)
1.	Construction Facilities					$ 15,000	$ 5,000	$ 8,000
2.	Removals					$ 250,000	$ 55,000	$ -
3.	Earthworks					$ 684,750		
4.	Sanitary Piping					$ 1,149,500	$ 2,802,500	$ -
5.	WWTP Accessories (Boat and Motor)					$ 8,500	$ -	$ -
6.	WWTP Air Piping System					$ 1,325,500	$ -	$ -
7.	Security Fencing					$ 10,000	$ -	$ -
8.	New Lagoon Construction					$ 1,699,225	$ -	$ -
9.	Existing Lagoon #1 Construction					$ 178,550	$ -	$ -
10.	Existing Lagoon #2 Construction					$ 178,550	$ -	$ -
				Construction Sub-Total	$ 8,370,075	$ 5,499,575	$ 2,862,500	$ 8,000
				Construction Contingency Allowance (+-20%)	$ 1,675,000	$ 1,100,000	$ 573,000	$ 2,000
				MMBR Allowance (Incl Contingency)	$ 6,984,650	$ -	$ -	$ 6,984,650
				Headworks, UV, & Efluent Pumping Building Allowance (Incl Contingency)	$ 10,964,520	$ 8,324,520	$ 1,440,000	$ 1,200,000
				Engineering Allowance	$ 2,800,000	$ 1,493,000	$ 488,000	$ 820,000
				Environmental Study Allowance	$ 120,000	$ 50,000	$ 25,000	$ 45,000
				Geotechnical Allowance	$ 85 000	$ 26 000	$ 26 000	$ 33 000
				Sub Total	$ 30,999,245	$ 16,493,095	$ 5,414,500	$ 9,092,650
				15% HST	$ 4,649,887	$ 2,473,964	$ 812,175	$ 1,363,898
				HST Rebate	-$ 3 321 348	-$ 1 767 117	-$ 580 125	-$ 974 213
				GRAND TOTAL (HST Incl.) :	**$ 32,327,784**	**$ 17,199,942**	**$ 5,646,550**	**$ 9,482,335**

*** Not Including Land Purchase**

Appendix C:

Mechanical Treatment Report (Englobe)

PROJECT TITLE: MECHANICAL WASTE WATER TREATMENT PLANT

PROJECT NO: P-0015962-0-01-001

CONCEPT STUDY REPORT

Greater Shediac Sewerage Commission (GSSC), NB

Prepared by:

Catherine Guilbeault, Jr. Eng.
Englobe - Geoenvironment

And:

Danny Latulippe, Jr. Eng.
Englobe - Geoenvironment

Approved by:

Pierre Plourde, P. Eng.
Crandall – Dir. Municipal Engineering

JUNE 18th , 2018

REVISION 01

DOCUMENT REVISION INDEX			
Nº	Date	Description	Prepared by:
00	2018-05-30	Issued for internal comments	CG/DL
01	2018-06-18	Issued for conceptual study	CG/DL

Project Team

Crandall Engineering Ltd.

Project Leader	Pierre Plourde, P. Eng.

Englobe Corp.

Benoît Lagarde, P. Chemist

Catherine Guilbeault, Jr. Eng.

Danny Latulippe, Jr. Eng.

Table of Contents

TABL S

A P NDICES

1 Projec Overview

Crandal s curren ly comple i g a Lon -Term Was ewater tra egy for the Greater Shediac Sewerage Com issio (GSSC), w ich inclu es an assessment of re tment t chnologies to service he ommiss on's current a d fut re n eds.

Englobe, w ich team h s consider ble expe ience in wast water tre tment f ci ities, as man at d by Cran all o co duct a review of mechan cal w ste wate treatment pla t tech olo ies nd to el borat a co cep -level desi n a d c st estimate Cl ss V per the As ociati n o the A vancement of Cost Engi ee ing (AA E) of a syste t at ould be implement in Shediac as an upgrad to the existin wa tewat r treat en aerated ago ns. The esul ing an lys s will allow Cr nda l to det rmine the approach to be use in t e m dification of the exist ng facility to eff cientl a commodate the prese t and future n eds of the Co mi sio .

A re iew of avai abl t eatment te hnolog es t e selec ion of he met od deemed optima a d the prelimina y des gn of t e selected treatm nt proces and main equipme t are furth r descri ed in t e follow ng section .

2 Cap-Bru é was ew ter p ant backgro nd

T e informat on provided in thi docume t i based on Crandall's e a l of Apr l 23, 20 8 an the additional documents:

- D awin showing t e existin W TP;
- Infl ent / Effluent sampling data from 2016/201 ;
- Environmental Risk Asse smen completed by Crandall, which i cl des additional infl ent samp in ata;

The e istin Cap- ru é Ro d Wastewa er Tre tment lant consist f t o aerated la oon followed by a polishin ce l. Prio t being disc arged to the nearby receivin w ter (a d eventual y to t e Northumbe lan Strait), the e fluen is disinfected by u tra iolet radiati n. Ba ed on process inf rmati n pr vide b Cra dall, the fac lit receives a total of 849 m^3/day [2. 4 MGD] of aw waste ater per ay as an a era e dail f ow rat . Asi e f om the wate or ginat ng from the resident al sectors within the service areas, the industr discharge nput does no exceed 5% of the to al dry wea her flow a d comes rimaril from se vice indus ries like hea th c nter, m tels p armacy and r st urant acco ding t t e E viron ental r sk assessme t con cted by Crandall in 2014.

As noted in Crandall's flow summary, it is expected that the future flows in fifty years will be in the order of 20 071 m^3/day [5.30 MGD].

3 Basis of design

3.1 Influent characteristics

3.1.1 Previous and actual conditions

The average and peak flows for the present wastewater treatment facility (2018) are presented in Table 1. These data will serve as design basis for Englobe's preliminary process engineering in this document:

Table 1 : Design Criteria for the Average and Peakflows

Flow type	m³/day	MGD
Average	8495	**2.24**
Peak	20 071	**8.60**

The loading influent wastewater characteristics, provided by Crandall are as follow:

- Avg $CBOD_5$ 52 mg/l [443 kg/day]
- Avg TSS 56 mg/l [473 kg/day]

As part of an Environmental Risk Assessment prepared by Crandall in 2014 on GSSC's Cap-Brulé Facility, Influent and Effluent samples were collected for further analysis. Data collected during the 2016 and 2017 period were analyzed by RPC Laboratory in Moncton.

Table 2 and Table 3 present the results obtained for each of the targeted contaminants done by RPC Laboratory in 2016 & 2017. The characterization has been conducted on the Influent and the Effluent. The table puts the average quarterly values in comparison:

Table 2 : Influent characterization results and comparison (years 2016 & 2017)

Evaluated parameter	Unit	Quarter	Year 2016			Year 2017			Avg Difference (%)
			Average	Min	Max	Average	Min	Max	
Carbonaceous Biochemical Oxygen Demand ($CBOD_5$)	mg/L	Q1	21	13	34	43	11	96	-104,76
		Q2	26	17	45	53	37	89	-103,85
		Q3	123	53	183	94	62	126	23,58
		Q4	64	36	97	52	23	99	18,75
Suspended Solids (SS)	mg/L	Q1	27	17	39	54	15	89	-100,00
		Q2	35	16	45	51	36	83	-45,71
		Q3	120	53	183	97	62	129	19,17
		Q4	68	36	97	57	26	93	16,18

Notes:

(1) : Full data available in Appendix A.

(2) : Q1(Jan-Mar) Q2(Apr-Jun) Q3(Jul-Sep) Q4(Oct-Dec)

(3) : Average of 6 samples per quarter.

Table 3 : Effluent characterization results and comparison (years 2016 & 2017)

Evaluated parameter#	Unit	Quarter	Year 2016			Year 2017			Avg Difference (%)
			Average	Min	Max	Average	Min	Max	
Carbonaceous Biochemical Oxygen Demand ($CBOD_5$)	mg/L	Q1	8	6	9	9	6	13	11,11
		Q2	9	6	14	7	6	10	-28,57
		Q3	6	6	7	10	6	13	40,00
		Q4	7	6	9	6	6	6	-16,67
Suspended Solids (SS)	mg/L	Q1	9	5	16	8	5	13	-12,50
		Q2	25	15	35	10	6	14	-150,00
		Q3	13	5	17	18	11	44	27,78
		Q4	9	5	16	7.5	5	11	-20,00
Total Ammonia Nitrogen (TAN)	mg/L	Q1	8.57	5.8	9.8	11	8.6	13	22,37
		Q2	6.85	5.3	8.1	8.7	5.8	12	21,26
		Q3	29.41	15.9	59	21.4	12	27	-207,31
		Q4	20	15	24	22.8	21	25	12,09
E. coli	MPN / 100mL	Q1					2	730	
		Q2		2	154		0	232	
		Q3		2	108		0	56	
		Q4		2	168		1	48	
Faecal strep	mg/L	Q1					6	1360	
		Q2					2	198	
		Q3					1	7.2	
		Q4					3	31.3	

Notes:
(1) : Full data available in Appendix A.
(2) : Q1(Jan-Mar) Q2(Apr-Jun) Q3(Jul-Sep) Q4(Oct-Dec)
(3) : Average of 6 samples per quarter.

3.1.2 Forecasted wastewater conditions

Below is a summary of the current available data for existing and forecasted flows and $CBOD_5$ and TSS loading. The new WWTP shall meet these estimated forecast assumptions. These values may be subject to changing depending of new outcome.

Table 4 : Forecasted values for entry volume and loading

	Existing					25 years					50 years				
	Flow	$CBOD_5$		TSS		Flow	$CBOD_5$		TSS		Flow	$CBOD_5$		TSS	
Month	m^3/d	kg/d	mg/l	kg/d	mg/l	m^3/d	kg/d	mg/l	kg/d	mg/l	m^3/d	kg/d	mg/l	kg/d	mg/l
January	10,220	378	37	406	40	13,851	1,032	74	1,060	77	20,905	2,302	110	2,330	111
February	10,805	284	26	402	37	14,436	937	65	1,056	73	21,491	2,207	103	2,326	108
March	9,586	318	33	414	43	13,217	972	74	1,067	81	20,272	2,241	111	2,337	115
April	12,255	352	29	386	32	15,886	1,006	63	1,040	65	22,940	2,276	99	2,309	101
May	13,128	584	45	584	45	16,759	1,238	74	1,238	74	23,814	2,508	105	2,508	105
June	10,217	450	44	534	52	13,848	1,103	80	1,187	86	20,902	2,373	114	2,457	118
July	6,956	663	95	586	84	12,403	1,643	132	1,566	126	22,984	3,548	154	3,471	151
August	5,977	718	120	727	122	11,423	1,699	149	1,708	149	22,005	3,603	164	3,612	164
September	5,069	530	105	573	113	8,700	1,183	136	1,226	141	15,755	2,453	156	2,496	158
October	4,925	244	50	321	65	8,556	897	105	975	114	15,611	2,167	139	2,245	144
November	5,682	401	71	398	70	9,313	1,054	113	1,051	113	16,368	2,324	142	2,321	142
December	7,121	399	56	349	49	10,752	1,052	98	1,002	93	17,806	2,322	130	2,272	128
Average	**8,495**	**443**	**52**	**473**	**56**	**12,429**	**1,151**	**93**	**1,181**	**95**	**20,071**	**2,527**	**126**	**2,557**	**127**

The data provided below will be used as a design basis for the dimensioning of the main treatment equipment:

Future (50 years projection)

- Average flow: 20,071 m^3/day (5.30 MGD)
- Peak flow: 56,000 m^3/day (14.8 MGD)

Loading

- Average $CBOD_5$ 126 mg/l
- Average TSS 127 mg/l

3.2 Regulatory Criteria and Design Effluent Objectives

To determine the effluent characteristics to be attained by the effluent of the treatment plant, federal and provincial regulations were considered. The environmental impact study performed in 2014 determined, based on these regulations and the latest NBDELG emitted COA, effluent discharge objectives (EDO) for substances of concern that can be found in the water. Table 5 presents these objectives.

Table 5 : Effluent discharge objectives

Contaminant	EDO
$CBOD_5$	**25 mg/L**
Total Suspended Solids	**25 mg/L**
Un-ionized ammonia	**1.25 mg/L**
Total Ammonia Nitrogen	**1.74 mg/L**
E. coli (2)	**200 MPN / 100mL**
pH	**6.5 to 9 (1)**

Notes: (1) CCME EQO for freshwater
(2) E. coli removal will be treated by UV disinfection of the effluent
That is not in the scope of the present study.

4 Review of available technologies

The project purpose is to evaluate the replacement of the actual aerated lagoons by a water treatment plant. The treatment plant should be composed of a primary treatment, including screening and grit removal, and a secondary/tertiary treatment process. The selected strategy as the secondary treatment is biological nutrient removal (BNR). This method is largely used in municipal wastewater treatment and can accommodate various conditions. BNR allows the removal of nitrogen and phosphorous compound in addition to largely decrease the water $CBOD_5$ loads by microbial degradation of waste.

Several configurations of BNR systems exist, but they all operate on the same concept: bioreactor tanks achieving nitrification and denitrification. The three more-common configuration of BNR systems are described in detail in this section: Activated sludge, sequencing batch reactor and moving bed biofilm reactor.

The principal advantage of these process compared to aerated lagoons is the diminution of the treatment facility footprint. The future of wastewater facilities is prone to converge toward this type of treatment where land is restricted.

4.1 Activated sludge

Activated sludge is the classical configuration of biological wastewater treatment system. Its most simple configuration consists of an aerated tank and a clarifier. The aerobic tank allows the oxidation of carbonaceous and nitrogenous matter and phosphorus removal by microorganisms' activities. The aerated wastewater will also form matter flocs that will be easy to settle in order to decrease the total suspended solids load of the water. Aeration and agitation (by air or mechanical agitator) must be maintained in this tank for the microorganisms to stay suspended in the wastewater and have access to the required dissolved oxygen for biodegradation. Once the required residence time in the tank is achieved, the mixture of wastewater and biological mass is transfer to a clarifier where the sludge is settled and the cleaned effluent is discharged. Part of the settled sludge is transfer back to the aerated tank to act as microbial inoculant for the new wastewater to be digested. Sludge that is not reuse is discharged from the clarifier and transfer to subsequent sludge management system.

Some activated sludge systems may have an additional tank which is not aerated. In this anoxic tank, nitrates produced by microbial aerobic activities are converted to gaseous nitrogen that can be discharged from the process.

4.2 Sequencing Batch Reactor (SBR)

Sequencing batch reactor is a variation of the activated sludge treatment plant in which all steps are performed in the same vessel. Usually, a plant is composed of more than one SBR operated in parallel. There are usually five steps in a SBR cycle: Filling of the tank with raw wastewater; reaction in aerobic condition; settling in anaerobic conditions; decantation and discharge; and idling.

The entire cycle is performed in the same tank. Aeration is started during reaction phase and is then stopped for the remaining of the cycle. Once the water has been decanted and effluent is discharged, part of the settled sludge is settled, and some is left in the tank for digestion of the next batch.

Advantage of the SBR is that no clarifier is usually needed if raw effluent loads in BOD or TSS are under 400 mg/L, which is the case for GSSC wastewater.

Technology enhancement

Because of the increasing popularity of SBR in high capacity wastewater treatment plant, new technologies are beginning to emerge in order to optimize this equipment. One example of these technology improvement is from a Canadian based business, Technologies Ecofixe©. The company specializes in the manufacturing of a system to be added to SBR or lagoons that allow partial fixation of the biological matter on a polymeric media and therefore extending the treatment capacity of a reaction tank without additional volume expansion.

4.3 Moving Bed Biofilm Reactor (MBBR)

The moving-bed biofilm reactor is a high efficiency biological treatment process acting has hybrid between processes using activated sludge and biofilms. The technology has been developed for the reduction of Biological Oxygen Demand (BOD) and Chemical Oxygen Demand (COD) in wastewater streams. The principle of the technology consists of an attached growth treatment method, in which diverse community of microorganisms, including those responsible for the treatment, grow on neutral HDPE carriers acting has a stable base. The carrier material is submerged in reaction tanks, whether in aerobic or anoxic condition, depending of the decontamination to be achieved, where the leachate is in constant movement to ensure good mixing (fluidization). The treatment is mainly provided by the fixed biomass on the carrier that develop due to the pollution charge in the leachate.

As for the active agent, these units serve as matrix on which the microorganisms proliferate and form a large protected biomass attachment area (biofilm) that will allow digestion of the water contaminants achieved by an intense biological activity. Since the microorganisms fixed to the media stay in the designated tank, no sludge recirculation is necessary. The attached growth media has a very high surface-to-volume ratio, allowing for a high concentration of biological growth to thrive within the internally protected areas.

The MBBR is a self-sustaining biological process, eliminating the need to periodically waste sludge and the requirement to supply a dilute return sludge to maintain a food-to-microorganism (F/M) ratio. The sloughed biomass from the attached growth media will remain suspended within the reactor and is continuously removed from the process by the existing flow stream, resulting in an operator free biological system. Sludge settling in this clarifier can directly be discharged to the following sludge management units and decanted effluent to the remaining of the treatment process.

The MBBR technology is a high efficiency solution for waste water treatment. Some of the advantages associated to this technology are enumerated below:

- Field Proven – high rate biological treatment process
- Higher concentration of biomass
- Quickly Responds to Load Fluctuations – Aggressive sloughing action enables the process to rapidly respond to variations in process load. This fixed biomass can quickly adapt to changes in loads and flows. In the event of a toxic shock, there is always a part of the fixed biomass which makes it possible to restart the bioreactors quickly.
- Resilient to Toxic Shocking – Fixed film process will slough off outer layer of dead bacteria and continue to produce more resistant new bacteria to meet the organic load.
- Small System Footprint – The fluidized fixed film reactor system is typically a fraction of the size of extended aeration system given an equivalent hydraulic load and concentration of COD/BOD.
- High Surface Area Fixed Film Biomedia – Suspended carrier elements designed for high rate fixed film biological treatment within a small footprint.
- No Sludge Bulking – The highly agitated process environment eliminates the buildup of biomass in the reactor to prevent sludge bulking.

- No Sludge Return – The fluidized fixed film reactor is a self-sustained biological process, eliminating the need to return a dilute activated sludge stream.

5 Selected technology

5.1 Treatment process

The challenge the GSSC faces is finding a process able to accommodate the projected volumes. In addition, the process must be able to achieve adequate performance in order to comply with regulations.

To meet the criteria presented in Section 3.2, the process should allow to considerably reduce the $CBOD_5$ concentration below 25 mg / L. With biological reactor technology, it will be possible to reach this value. In addition, the use of a 2-phases biological reactor, aerobic and anaerobic, will remove nitrogen which is also a substance to monitor. Finally, using a 2-phases reaction technology, will reduce the microbial load 15-20% and thus increase the disinfection. As mandated, Englobe has developed a preliminary conceptual design for the WWTP on so-called mechanical technologies in order to compare it to the performance of a lagoon enlargement.

5.1.1 Selected Process Description

Following a review of the available technologies applicable to the GSSC and the criteria that the WWTP must meet, Englobe advices to proceed with the implementation of a MBBR system. This technology is known to achieve concentrations of $CBOD_5$ and TSS of less than required by the regulation.

In our opinion, a Suspended Media Bioreactor (MBBR) treatment process will provide several advantages to the foreseen mechanical plant implementation. The MBBR technology is suitable for this project and can be added as an up-grade to existing biological wastewater plants.

One major advantage of the MBBR process, when compared to an SBR process, is related to a much more compact footprint. Based on the proposed design, the MBBR bioreactors would require 80% less volume than the compared technology. The following table shows the difference in volume and equipment between these 2 systems:

Table 6 : MBBR vs SBR

Parameter	MBBR	SBR
Global effective volume	2130 m^3	12000 m^3
Number of bioreactor	4 (1)	3
Volume of bioreactor	600 m^3 & 465m^3 (2)	4000 m^3
Hydraulic retention time (HRT)	↓	↑

Notes: (1) 2 trains in parallel, each designed for 50% of the future flow rate foreseen in 50 years
(2) $CBOD_5$ reduction reactors are 600m^3 and ammonia reduction reactors are 465m^3

Furthermore, the MBBR process has been selected giving the fact that SBR will be much more complex to operate. SBR monitored operational parameters would evolve over time as the flow rate and the wastewater loadings fluctuates, making these bioreactors more complex to adjust and operate.

Also, since it is planned that the forecasted volume will considerably grow in the near future, a fixed biomass treatment system is well indicated. Fixed biomass allows the treatment of a larger pollution loading per tank volume than other biological treatments such as activated sludge. This technology is robust enough to hold up to high and variable contamination loads, such as those faced by municipal wastewater facilities

The following flowchart represents the different steps followed by the wastewater after primary treatment

5.1.2 Equalization tank

Before the MBBRs, an equalization tank will be installed in order to maintain a constant flow in the treatment units. From the forecasted flow value provided by Crandall Engineering, it was determined that a 1000 m^3 basin will be sufficient to avoid overflow between the primary treatment system and the MBBR. The tank will also allow partial TSS removal by decantation. Sedimented solids will need to be pumped and transfer to one of the sludge transfer tanks to be dehydrated than disposed. Since this tank will likely be located outside, a membrane can be installed on its surface to counter the nuisance caused by odors emanating from contaminated water before it is treated.

5.1.3 Neutralization

Since the MBBR treatment process includes an anoxic phase, it is possible that the pH of the effluent is changed and the water at the outlet of the treatment is alkaline cause the proliferation of anaerobic bacteria. To ensure compliance with the pH limits established by the authority having jurisdiction, a neutralization pond will be installed downstream of the treatment process. Acid addition will be made to this tank before the water is discharged to the environment or directed to the ultraviolet disinfection facility.

5.2 Clarification and Sludge management

Following the MBBR, a clarification unit is required in order to remove all particulate matter at the effluent of the bioreactor. The clarification system should consist of dissolved air floatation (DAF) units preceded by coagulation and flocculation basins, with effective volumes of respectively 22m^3 and 42m^3. Water coming out of the MBBRs will be forwarded to the basins were flocculant and coagulant will be added before being redirected to the DAF units that has a combined capacity of 19 200 m^3/day. Coagulant will be chosen in order to allow precipitation of residual phosphorous following the biological treatment. Removal of this nutrient will be performed by solids sedimentation in the DAF units. The DAF clarifier consists of a fiber reinforced plastic(FRP) tank with lamella plates where air bubbles rising bring suspended flocculated matter at the surface of the liquid. Floating matter is discarded by the surface skimmer to the sludge transfer tank. Denser matter is sedimented on the lamella and at the bottom of the tank from where it is pumped to the sludge transfer tank. Clarified water is then transfer to the neutralization tank.

Since the sludge produce by the DAF clarifier has a high water content, it is necessary to partly dehydrate it. From the sludge tank, it will be directed to two (2) screw presses. Each press has a treatment capacity of 320kg of dehydrated sludge per hour. A sludge accumulation basin made of concrete may be necessary upstream from the presses. This basin should be aerated in order to avoid production of odors from microbial anaerobic digestion of the DAF sludge. The filtrate from the presses is then transfer to the neutralization tank.

5.3 Expected water composition

The treatment process presented above is expected to lower the contaminant concentration in water to an acceptable level. The following table present the expected final concentrations after the treatment.

Table 7 Effluent expected composition

Contaminants	Final effluent concentration
$CBOD_5$	< 20mg/L
TSS	< 20mg/L
TAN	< 1 mg/L
Phosphorus	< 0.5 mg/L

5.4 Odor control

Odors produced by wastewater treatment are mainly due to production of sulfurous gas, such as H_2S. In the plant, the major concern for odors will be the outside installations. If these basins cause a concern for the close-by inhabited area, it is proposed to install a floating membrane to maintain biogas produced by microbial activity.

In general, odor production from MBBRs should not be more of a concern than those produced by the existing wastewater facility.

6 Cost estimate

Englobe is responsible for developing the Capex direct cost for the major process equipment of the water treatment plant. Excluded from Englobe's responsibility are the primary treatment and disinfection systems (by Crandall).

The Direct Costs were based on engineering take offs and scope and quantity reviews were performed with engineering to confirm that the scope of work was entirely covered.

The cost estimate was based on a Class V type estimate, per the Association for the Advancement of Cost Engineering (AACE), methodology as defined by the type and quantity of engineering deliverables produced to support the estimate. The expected order of accuracy is in the range of ± 50%.

The estimate shown in Table 6 is based on budget quotations received from manufacturer, in Canadian dollars.

Table 8 : Summary of major process equipment cost

Major equipment description	QTY	COST ($CAD)
GSSC Waste water treatment plant	**1 LOT**	**3,4 M$**
Biological Secondary Treatment (MBBR)	**1 LOT**	**INCLUDED**
Bioreactor concrete tanks ($CBOD_5$ removal) (13.5m long. x 6.8m wide x 7.3m deep)	2	By Civil
Bioreactor concrete tanks (nitrification) (10.5m long. x 6.8m wide x 7.3m deep)	2	By Civil
Aeration grids incl. medium bubble diffusers	4	Incl.
Media retention screens (outlet)	2	Incl.
Media retention screens (intermediate)	2	Incl.
Media retention screens (instrumentation)	2	Incl.
HPDE Media (750m^2/m^3)	1150m^3	Incl.
Isolation valves and vacuum breakers	1 LOT	Incl.
Instrumentation	1 LOT	Incl.
Clarification system	**1**	**INCLUDED**
Coagulation basin in concrete (3.12m long. x 2.34m wide x 3.3m deep)	2	By Civil
Flocculation basin in concrete (4.46m long. x 3.12m wide x 3.3m deep)	2	By Civil
Clarification system (continued)	**1**	**INCLUDED**
Coagulation agitator/mixer	2	Incl.
Flocculation agitator/mixer	2	Incl.
Dissolved air floatation (DAF) clarifier	2	Incl.
Piston air compressor	1	Incl.
Sludge transfer tank (4.5m^3 capacity)	2	Incl.

Major equipment description	QTY	COST ($CAD)
Sludge transfer pump	3	Incl.
Valves	1 LOT	Incl.
Instrumentation	1 LOT	Incl.
Sludge Dehydration	**1 LOT**	**INCLUDED**
Sludge storage tank in concrete (13.6m long. x 6.8m wide x 7.3m deep)	1	By Civil
Aeration grid with medium bubble diffusers	1	Incl.
Rotary blower with acoustic housing	1	Incl.
Multi disc screw press	2	Incl.
Control panel	1	Incl.
Valves	1 LOT	Incl.
Instrumentation	1 LOT	Incl.
Anti-Foaming Agent Dosing System	**1 LOT**	**INCLUDED**
Anti-foaming agent dosing system including one dosing skid assembly, metering pump, valves and instruments	1 LOT	Incl.
Coagulant Dosing System	**1 LOT**	**INCLUDED**
Coagulant dosing system including one dosing skid assembly, metering pump, valves and instruments	1 LOT	Incl.
Polymer Dosing System	**1 LOT**	**INCLUDED**
Dry polymer preparation dosing system including one dry feeder with hopper, heating element, regenerative blower, electrical panel, mechanical agitator, preparation tank, storage tank, three metering pumps, dosing skid assembly, valves and instruments	1 LOT	Incl.
Sludge Polymer Dosing System	**1 LOT**	**INCLUDED**
Dry polymer preparation dosing system including one dry product aspirator, mechanical agitator, preparation tank, two metering pumps, dosing skid assembly, valves and instruments	1 LOT	Incl.
Control Panel	**1 LOT**	**INCLUDED**

Major equipment description	QTY	COST ($CAD)
One integrated control panel as per NEMA standards, 600/3/60 including PLC and HMI	1 LOT	Incl.
Auxiliary	**1 LOT**	**EXCLUDED**
Influent pumping skid	1 LOT	Excluded
Treated effluent pumping skid	1 LOT	Excluded
Dewatered sludge bins	1 LOT	Excluded
Power supply	1 LOT	Excluded
Building	1 LOT	Excluded

Conclusion and Recommendation

At this stage, we recommend using a MBBR (Moving Bed Bio-Rector) process for the GSSC wastewater treatment facility. This technology has a lower maintenance cost, a smaller footprint and is easier to operate compared to the SBR option previously mentioned to Crandall.

Englobe recommends pursuing the project to the feasibility stage level and project development. During the detailed engineering stage, additional assessments and activities should be considered, notably the following:

1. Feasibility study (Class 4 cost estimate including OPEX)
2. Risk assessment;
3. Schedule;
4. Impact study (social, environment, air, water)
5. Geotechnical survey;
6. Permitting;
7. Civil and landscape cost estimate;
8. Building cost estimate;
9. Communication plan.

Moreover, the system is designed to allow parallel operation of the two distinct equipment trains (MBBR > DAF clarifier > screw press). As a first project phase, the installation of only one of these two equipment trains may be considered. This arrangement should suffice the actual GSSC needs. However, if this option is retained, it should be considered that, with only one equipment train, the system redundancy is lost and, in the case of an emergency maintenance that should affect one of the equipment, the entire plant operation may be stopped.

References

[1] UNITED STATES ENVIRONMENTAL PROTECTION AGENCY (1999) Wastewater technology fact sheet: Sequencing Batch Reactor. EPA 932-F-99-073.

[2] GOVERNMENT OF CANADA – JUSTICE LAWS (2012) Wastewater systems effluent regulations. Source: http://laws-lois.justice.gc.ca/eng/regulations/SOR-2012-139/FullText.html

[3] UNITED STATES ENVIRONMENTAL PROTECTION AGENCY (2013) Emerging Technologies for Wastewater Treatment and In-Plant Wet Weather Management. EPA 832-R-12-011.

[4] UNITED STATES ENVIRONMENTAL PROTECTION AGENCY (2008) Municipal Nutrient Removal Technologies Reference Document. EPA 832-R-08-006.

[5] CRANDALL ENGINEERING (2014) Environmental Risk Assessment: GSSC (Cap-Brulé) Wastewater treatment plant – Final Report.

[6] AACE INTERNATIONAL, (latest revision) Recommended Practice No.18R-97: Cost Estimate Classification System – As Applied in Engineering, Procurement, and Construction for the Process Industries.

[7] MABAREX (2018) Mechanical Plant Concept Study – Request for proposal. Ref.: Nº 6481, Revision A.

APPENDIX A – Waste water analysis raw data

2016 Cap Brulé WWTP - Lab Results

Date	Influent		Effluent			
	$CBOD_5$	SS	$CBOD_5$	SS	TAN	E. Coli
5-Jan	16	17	7	5	5.80	
20-Jan	17	19	8	16	7.70	
2-Feb	23	18	9	8	9.00	
17-Feb	16	36	8	7	9.80	
2-Mar	31	30	9	7	6.70	
16-Mar	34	39	6	5	7.60	
29-Mar	13	31	9	12	8.60	
average	21	27	8	9	8.57	
12-Apr	17	20	8	23	5.30	
27-Apr	16	25	6	18	5.50	2
11-May	21	24	6	15	8.10	6
25-May	20	28	9	24	8.00	16
8-Jun	37	68	12	32	6.70	154
16-Jun						56
22-Jun	45	43	14	35	7.50	6
average	26	35	9	25	6.85	
6-Jul	53	43	6	9	15.90	6
19-Jul	127	109	6	5	23.00	2
3-Aug	183	221	6	14	26.00	4
17-Aug	175	159	6	16	59.00	108
31-Aug	106	67	7	15	26.00	2
14-Sep	86	103	6	17	29.00	2
29-Sep	133	141	6	12	27.00	2
average	123	120	6	13	29.41	
19-Oct	86	129	6	5	24.00	2
26-Oct	41	39	6	7	23.00	168
9-Nov	97	107	6	8	22.00	
23-Nov	46	35	6	6	19.00	
7-Dec	36	27	9	16	17.00	
20-Dec	76	71	7	9	15.00	
average	64	68	7	9	20	

- Effluent standards are: (as per DELGNB CofA S-2627)

$CBOD_5$ and SS - quaterly average ≤ 25mg/l (SS exception >25mg/l, Jul-Oct.)
E.Coli ≤ 200 MPN/100ml - Only from May 1st to October 31st

- Legend: **SS**: Suspended Solids, **$CBOD_5$**: Carbonaceous Biochemical Oxygen Demand,
TAN: Total Ammonia Nitrogen,

These analyses are conducted by RPC Laboratories - Moncton

2017 Cap Brulé WWTP - Lab Results

	Influent		Effluent						
Date	$CBOD_5$	SS	$CBOD_5$	SS	TAN	before UV E. coli	after UV E. Coli	before UV faecal strep	after UV faecal strep
05-Jan	96	89	6	8	13.00		366		172
17-Jan	19	34	11	10	13.00		730		1360
01-Feb	11	22	6	7	10.30	252	6		18
15-Feb	55	73	6	5	11.10		10		18
01-Mar	18	15	8	5	11.70		12		22
14-Mar	58	72	13	13	8.60		2		14
30-Mar	45	72	10	10	9.60		2		6
Average	43	54	9	8	11.04				
12-Apr	37	45	10	8	12.00		232		198
27-Apr	45	36	6	9	9.00		24		44
10-May	48	43	6	10	7.40		122		42
24-May	89	83	6	14	5.80	7200		2200	
29-May							4		2
Date	$CBOD_5$	SS	$CBOD_5$	SS	TAN	before UV E. coli	after UV E. Coli	before UV enterococci	after UV enterococci
06-Jun	47	49	6	11	6.80		2		6.2
20-Jun			6	6	11.20		0		10.3
Average	53	51	7	10	8.70				
05-Jul	126	123	12	20	12.00		1		3.0
19-Jul	75	62	9	11	17.00		2		3.0
02-Aug	99	80	6	24	20.00		0		6.2
16-Aug	96	99	6	29	26.00		0		1.0
30-Aug	62	104	10	44	27.00		0		2.0
12-Sep	97	79	13	35	24.00		0		5.2
27-Sep	102	129	11	30	24.00		56		7.2
Average	94	97	10	18	9.57				
12-Oct	23	26	6	6	25.00		1		3.0
25-Oct	48	67	6	8	23.00		3		3.0
08-Nov	40	45	6	5	22.00		48		31.3
22-Nov	99	93	6	11	21.00		12		11.3

- Effluent standards are: (as per DELGNB CofA S-2627)

$CBOD_5$ and SS - quaterly average ≤ 25mg/l (SS exception >25mg/l, Jul-Oct.)
E.Coli ≤ 200 MPN/100ml - Only from May 1st to October 31st

- Legend: **SS**: Suspended Solids, **$CBOD_5$**: Carbonaceous Biochemical Oxygen Demand,
TAN: Total Ammonia Nitrogen,

These analyses are conducted by RPC Laboratories - Moncton

E coli results from November to April do not have to be reported
Faecal Strep + Enterococci results do not have top be reported
May 24 - Operator mistakenly took sample before UV system. Sample was retaken May 29 after UV system.

Appendix D:

Photos from Site Review

Figure 1: Screw Pump Building - Exterior

Figure 2: Screw Pump Motors and Mounting

Figure 3: Screw Pump Auger

Figure 4: Screw Pump Wet-Well

Figure 5: LS No.24 Exterior

Figure 6: Pre-Treatment Building - Equipment

Figure 7: Lagoon Cells No.1 and No.2

Figure 8: Blower Building - Exterior

Figure 9: Blower Building - Interior

Figure 10: Blower Building - Blowers

Figure 11: Maintenance Garage - Exterior

Figure 12: UV Building - Interior