Niagara Region

REGIONAL MUNICIPALITY OF NIAGARA
SOUTH NIAGARA FALLS WASTEWATER SOLUTIONS

V3.11 – WWTP Design Basis

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V3.11.1

REGIONAL MUNICIPALITY OF NIAGARA
SOUTH NIAGARA FALLS WASTEWATER SOLUTIONS

WWTP Design Basis

Design Basis – New WWTP

Regional Municipality of Niagara

South Niagara Falls Wastewater Solutions Schedule C Class Environmental Assessment and Conceptual Design

TM No. 1 Design Basis

March 2022

SUBMITTED BY CIMA CANADA INC.
5935 Airport Road
Mississauga, ON L4V 1W5
T 905 695-1005
cima.ca

South Niagara Falls WWTP Class Environment Assessment and Conceptual Design
TM No. 1 Design Basis

Project T001140A

PREPARED BY:

Mina Yousif, M.Eng., P.Eng., PMP

VERIFIED BY:

Troy Briggs, M.Eng., P.Eng.

CIMA+
5935 Airport Road, Unit 500
Mississauga, Ontario L4V 1W5

March 2022

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1 Introduction

1.1 Background

The Regional Municipality of Niagara (Niagara Region) completed a Water and Wastewater Master Servicing Plan (Master Plan) in 2017 that provided a long-term planning strategy to address the water and wastewater system needs to the year 2041 (GM BluePlan, 2017). The Master Plan recommended a combination of solutions for meeting future needs, including improving the existing sewage collection systems, and construction of a new wastewater treatment plant (named South Niagara Falls WWTP) to service growth in south Niagara Falls in two stages:

- Stage 1: Provide a capacity of 30 megaliters per day (MLD), including approximately 15 MLD from the existing Niagara Falls WWTP, which currently services the existing developed South Niagara Falls area, and approximately 15 MLD from new growth in that area;
- Stage 2: Provide a capacity increase to 60 MLD to accommodate future servicing to full build-out capacity.

The 2017 Master Plan was completed under the Environmental Assessment Act in accordance with Phases 1 and 2 of Municipal Class Environmental Assessment (EA) requirements (2000, as amended in 2007, 2011 and 2015). The Master Plan concluded that a Schedule “C” Class EA study is required to address Phases 3 and 4 requirements of the Municipal Class EA planning process.

GM BluePlan, in association with CIMA+, has been retained by the Region to complete the Schedule “C” Class EA study and Conceptual Design for the proposed South Niagara Falls WWTP (SNF WWTP). The Class EA study will present development and evaluation of alternative design concepts for the preferred solution including their associated environmental impacts and proposed mitigation measures.

1.2 Purpose of TM No. 1

The objective of this technical memorandum (TM No. 1) is to establish the design basis for the proposed Stage 1 30 MLD South Niagara Falls WWTP, which considers various factors with respect to population projections, wastewater flows and loadings, and effluent objectives and limits.

The design basis will be used to develop and evaluate alternative design concepts as part of Phases 3 and 4 of the South Niagara Falls WWTP Class EA Study.

2 Historical Data Review

This section provides historical data review of the existing Niagara Falls WWTP including the raw wastewater flow, characteristics, loading and performance data. The purpose is to help develop and refine the design basis for the proposed SNF WWTP in consideration that approximately one-third of the new plant flow is currently tributary to the existing Niagara Falls WWTP.

In addition to servicing the South Niagara Falls area, the proposed SNF WWTP will also accept centrate loading from the Region’s nearby Garner Road Biosolids Facility for treatment. Therefore, the Garner Road historical centrate flow and loading data was also reviewed.

2.1 General Description

The existing Niagara Falls WWTP is a rotating biological contacting (RBC) plant providing wastewater treatment to the City of Niagara Falls, and the Town of Niagara-on-the-Lake. The plant has a current rated average day flow (ADF) capacity of 68.3 MLD, a peak dry weather flow capacity of 136.4 MLD and a peak wet weather flow capacity of 205.0 MLD.

Wastewater treatment processes include screening and grit removal, primary treatment, secondary treatment, phosphorus removal, and effluent disinfection (chlorination/dichlorination) prior to discharging to Ontario Power Generation Canal. Ferric chloride is added upstream of the secondary clarifiers for phosphorus removal.

Raw sludge is anaerobically digested, dewatered via an on-site centrifuge, with biosolids cake being trucked from the site for further processing by an external contractor. Dewatering centrate is returned to the plant headworks for treatment.

2.2 Historical Flows

Table 1 presents a summary of the historical recorded raw wastewater flows to the existing Niagara Falls WWTP over the 3-year review period of end of 2017 to early 2020. Over the review period, the ADF was 40 MLD, or approximately 60% of the plant rated ADF capacity of 68.3 MLD. The historical maximum day flow (MDF) and peak hour flow (PHF) represent a peak factor of 3.1 and 4.1, respectively. The peaking factors are considered high for a medium sized plant like the Niagara Falls WWTP, indicative of high infiltration/infow (I/I) experienced in the plant service area.

The historical annual average per capita flow was 285 L/cap/d, which is within the typical municipal per capita flow design range of 225 to 450 L/cap/d (MECP, 2008).

Table 1 Historical Flows to the Existing Niagara Falls WWTP (2018-2020)

Parameters	Influent Flow (MLD)	Peak Factor
Rated Capacity	68.3	
Average Day Flow (ADF)	40	-
Maximum Day Flow (MDF) (99.5th percentile)	124	3.1
Peak Hour Flow (PHF) (99.5th percentile)	164	4.1
Average Per Capita Flow (L/cap/d)	285	-

Notes:

1. Based on historical average day flow and the current total service equivalent population (i.e. sum of residential and employment pop.) of 140,000 provided in Planning Projection memo (GM BluePlan, 2020).

The historical average daily influent flow for the last 3 years (end of 2017 to early 2020) are graphically illustrated in Figure 1.

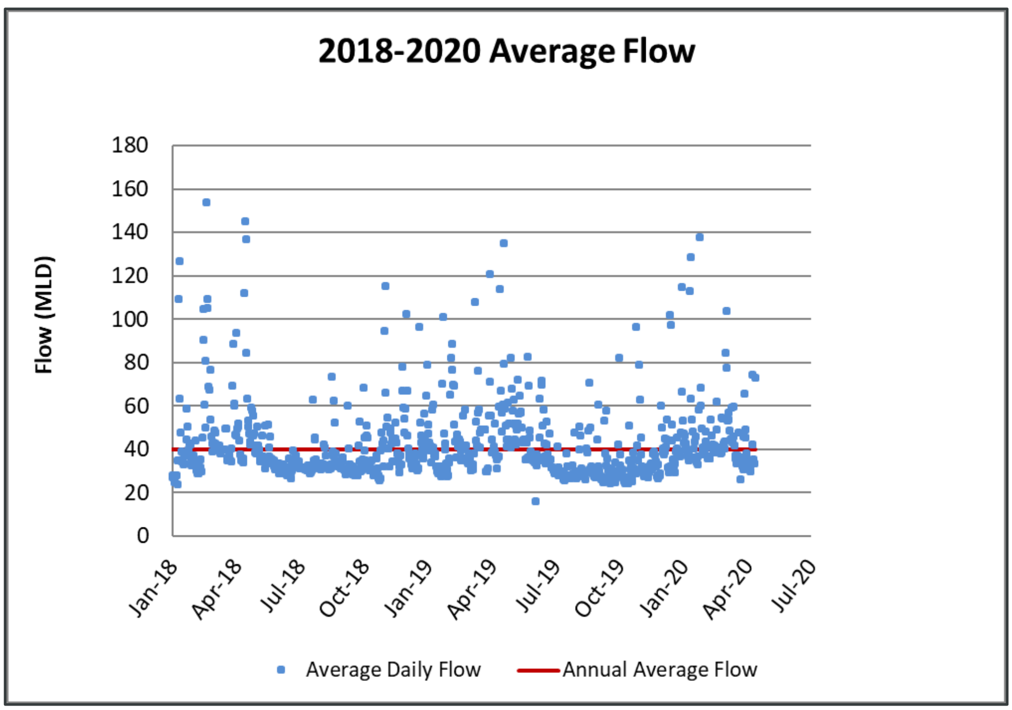

Figure 1 Average Daily Flow to the Existing Niagara Falls WWTP (2018 – 2020)

2.3 Historical Raw Wastewater Characteristics and Loadings

Historical raw wastewater concentration data for the last two years (2018 to 2019) were analyzed with flow data to establish the current plant loadings at the existing Niagara Falls WWTP. Table 2 presents a summary of historical raw wastewater concentrations and loadings for biochemical oxygen demand (BOD_5), total suspended solids (TSS), total phosphorus (TP), and total Kjeldahl nitrogen (TKN).

The historical raw influent wastewater can be characterized as medium strength with respect to BOD_5, TSS, and TKN, and low strength with respect to TP (Metcalf & Eddy, 2003). Historical per capita loadings are similar to typical design values for all parameters except TP. This is consistent with the medium strength of the wastewater and the relatively low average per capita flows.

Table 2 Niagara Falls WWTP Historical Raw Wastewater Loadings (2018- 2019)

Parameter	Total Average Day Load, kg/d [1]	Total Maximum Month Load kg/d [1], (PF)	Average Concentration, mg/L [2]	Estimated Per Capita Contribution, g/cap/d [3]	Typical (Range) Per Capita loading, g/cap/d [4]
BOD_5	9,310	14,230 (1.4)	220	67	75 (70-110)
TSS	12,310	18,500 (1.5)	290	88	90 (60-115)
TP	190	250 (1.3)	4.5	1.4	2 (2-5)
TKN	1,750	2,430 (1.2)	41	12.5	13 (9-14)

Notes:

1. Loadings for raw wastewater only (excluding centrate load recycled from the on-site centrifuge).
2. Calculated as historical average load of raw wastewater divided by the historical average day flow.
3. Based on historical average load (without centrate) divided by the current total service equivalent population (i.e. sum of residential and employment population) of 140,000 provided in Planning Projection memo (GM BluePlan, 2020).
4. Typical per capita loadings adapted from Metcalf & Eddy (2003).

Figure 2 to Figure 5 present the monthly average influent loadings for BOD_5, TSS, TKN, and TP, respectively. Over the 2-year review period, the raw wastewater loadings for all parameters have slightly increased with population increase. The loadings for all parameters are generally higher in summer time than in winter time, likely due to the added population from tourism during summer season in Niagara Falls.

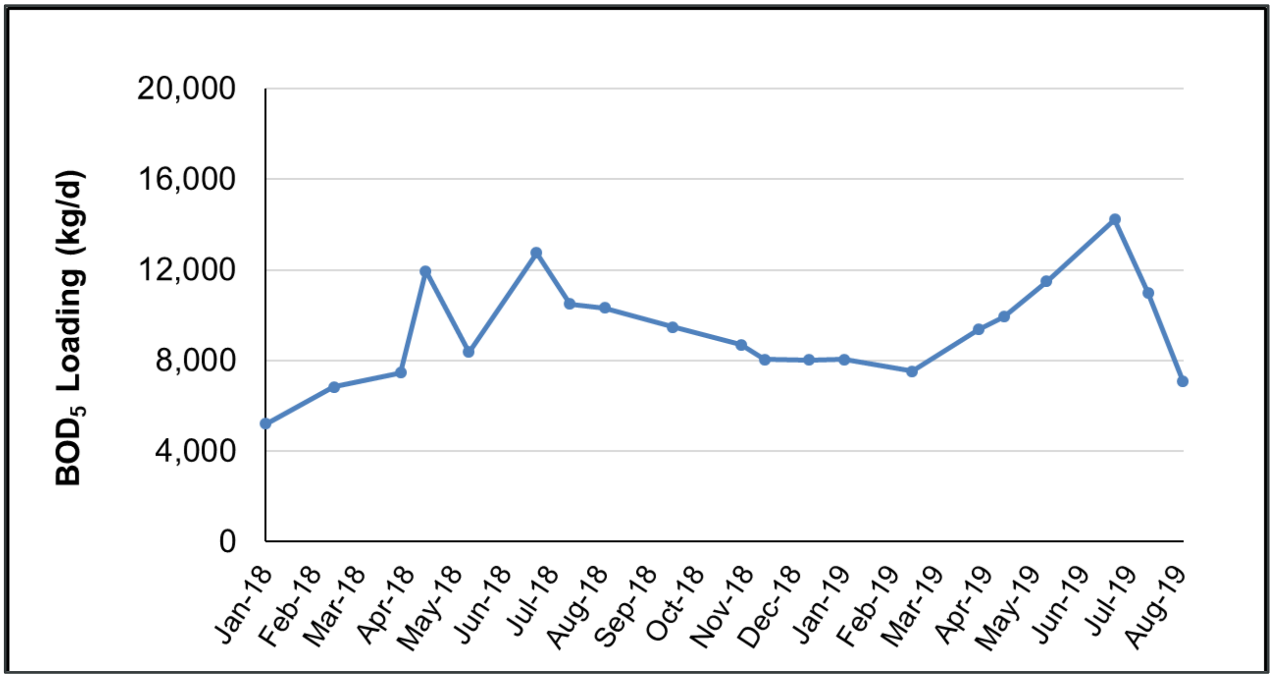

Figure 2 Niagara Falls WWTP Raw Influent BOD_5 Loading (2018 – 2019)

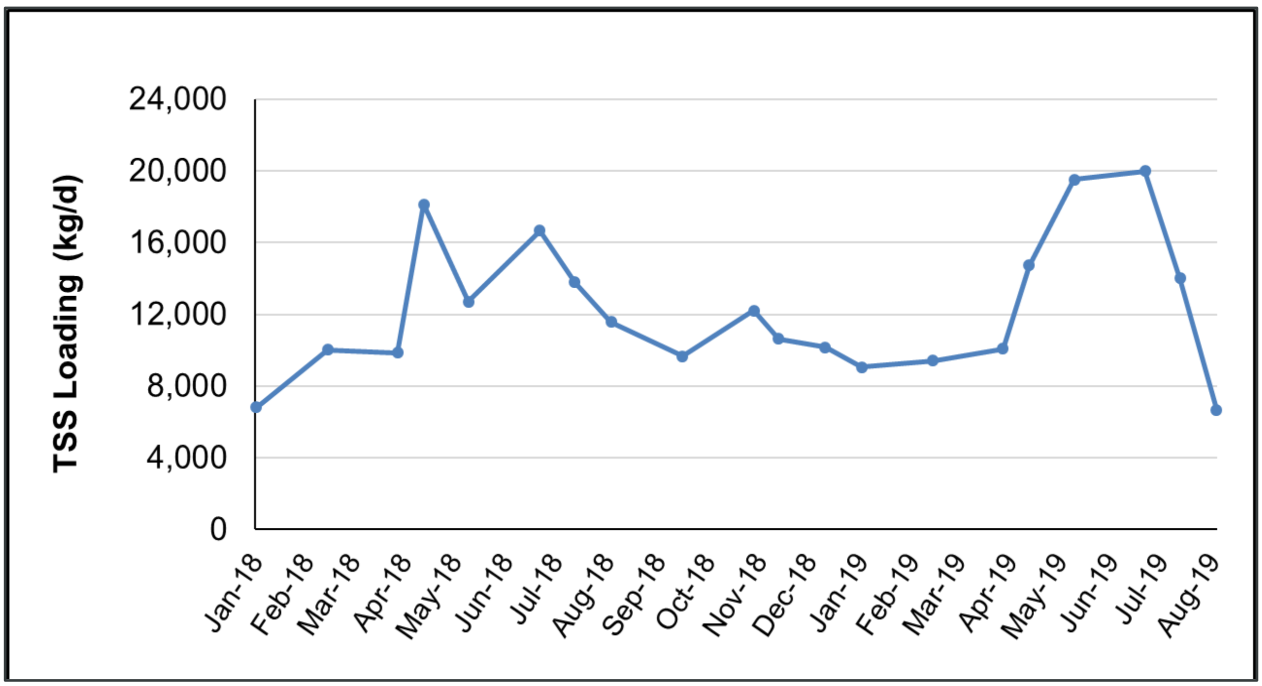

Figure 3 Niagara Falls WWTP Raw Influent TSS Loading (2018 – 2019)

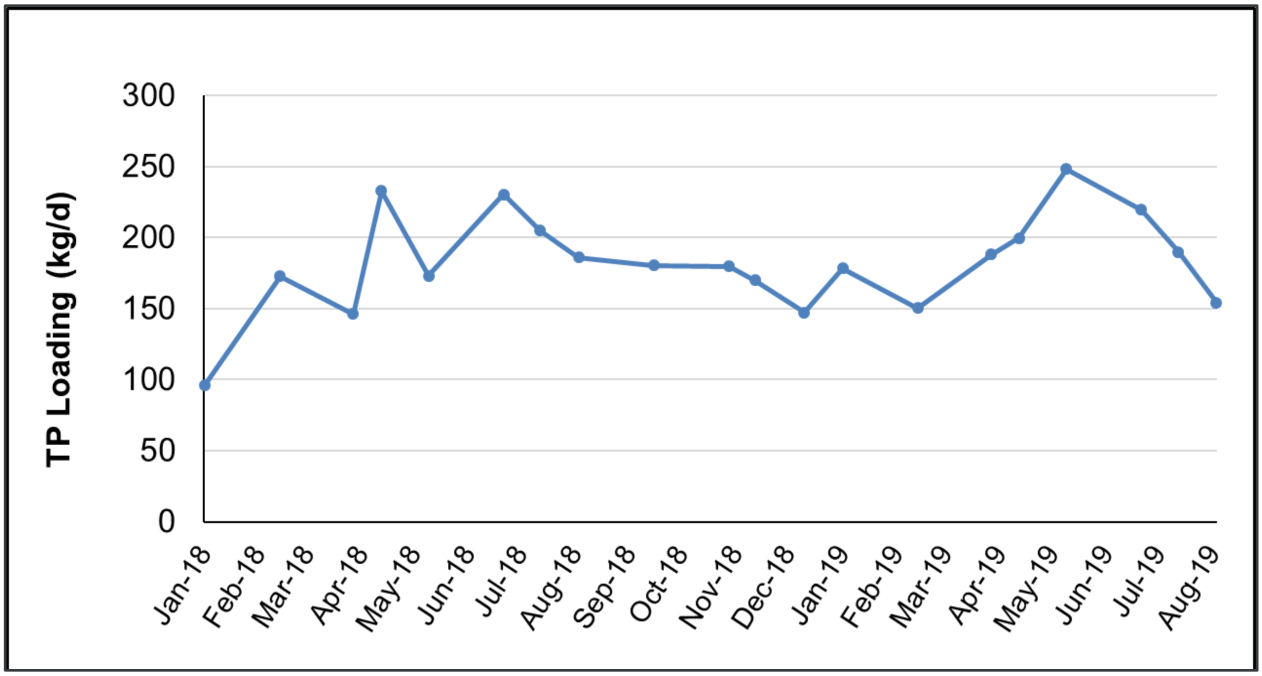

Figure 4 Niagara Falls WWTP Raw Influent TP Loading (2018 – 2019)

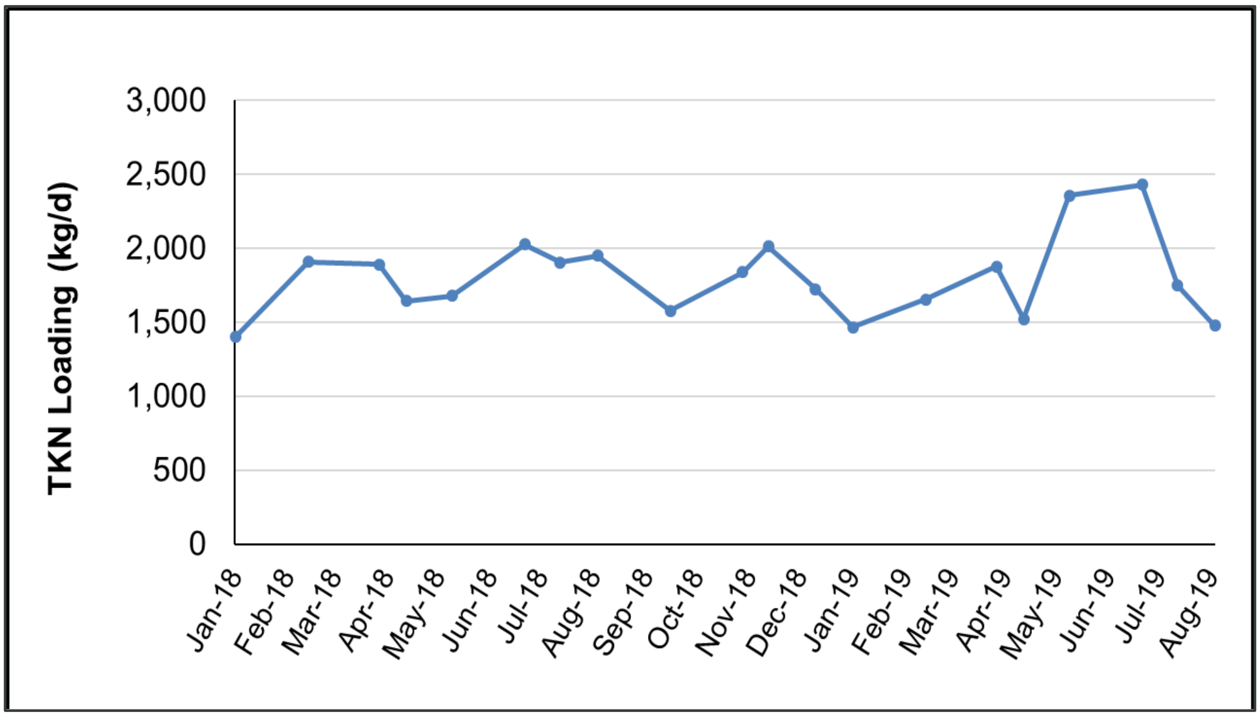

Figure 5 Niagara Falls WWTP Raw Influent TKN Loading (2018 – 2019)

2.4 Garner Road Centrate Flow and Characteristics

The Garner Road Biosolids Facility is located northeast of the intersection of Chippawa Creek Road (Niagara Regional Road 63) and Garner Road in the City of Niagara Falls. The facility receives liquid biosolids from several WWTPs in the Region for dewatering and/or management. The dewatered biosolids are transported to N-Viro, a biosolids processing facility in Thorold, Ontario to produce fertilizer for land application. Dewatering centrate is currently sent to the collection system and conveyed with raw sewage to the Niagara Falls WWTP for treatment. With the construction of the new SNF WWTP, it was proposed that the centrate from the Garner Road Biosolids Facility be treated at the new plant.

The Garner Road Biosolids Facility currently has two (2) centrifuges in operation and is adding a third centrifuge in the future, each rated approximately 2 dry tonnes/hr or 20 L/s at 3% total solids. The centrifuges are operated 8 hours per day and 7 days per week. Table 3 presents a summary of the historical average centrate flow during 2017 to 2019.

Table 3 Garner Road Biosolids Facility Annual Average Centrate Flow (2017-2019)

Year	Annual Average Flow
2017	1.08 MLD
2018	1.04 MLD
2019	1.00 MLD
Average	1.05 MLD

The historical centrate quality data from the Garner Road Facility was not available. To estimate the current loading of the Garner Road centrate, the existing Niagara Falls WWTP historical centrate characteristics data were reviewed as a reference and compared with the typical design values (Metcalf & Eddy, 2003), as summarized in Table 4.

Table 4 Niagara Falls WWTP Historical Centrate Characteristics (2017-2019)

Parameter	Historical Average Concentration (mg/L)	Typical Value (mg/L) (2)
BOD_5	250 (1)	500-1,000
TSS	380	1,000-2,000
TP	20	20-75
TKN	490	700-800
Note: (1) Based on historical data on November 27, 2018. (2) Based on Metcalf & Eddy, 2003.		

It is noted that the historical centrate concentrations for all the parameters are relatively low, specifically those for TKN, as compared to the typical design values. The low centrate TKN concentration is likely a result of the reduced anaerobic digester capacity currently in operation at the Niagara Falls WWTP (i.e. Primary Digester No. 1 out of service since June 2014 and both

Primary Digesters No. 1 and No. 2 out of service since July 2017). As a result, the historic centrate concentration data is not considered representative of a well operating anaerobic digestion facility.

To be conservative, typical design centrate concentrations were used to estimate the historical centrate loading from the Garner Road Biosolids Facility. The estimate results are summarized in Table 5.

Table 5 Garner Road Biosolids Facility Historical Centrate Loadings Summary (2017- 2019)

Parameter	Average Concentration (mg/L) [1]	Average Load (kg/d) [2]
BOD_5	750	790
TSS	1,500	1,580
TP	50	53
TKN	800	840
Notes: (1) Typical centrate concentrations adapted from Metcalf & Eddy (2003). (2) Based on historical centrate average flow of 1.05 MLD (2017-2019).		

3 Hauled Waste

Hauled waste will be received at the new SNF WWTP. With over eight (8) WWTPs available in the Region to accept hauled waste, it will be difficult to predict the amount of hauled waste for the new SNF WWTP. For design purposes, a daily hauled waste flow average of 0.15 MLD is assumed. Daily flow amounts to approximately ten (10) loads with each truck load being 15 m^3. Typical hauled waste concentrations and associated loads are summarized in Table 6.

Table 6 Hauled Waste Typical Concentrations and Loading

Parameter	Average Concentration (mg/L) [1]	Average Load (kg/d) [2]
BOD_5	7,000	1,050
TSS	15,000	2,250
TP	267	40
TKN	700	105
Notes: (1) Typical based on Table 3-8 US EPA Handbook – Septage Treatment and Disposal, 1984 (2) Based on project hauled waste flow of 150 m^3/d		

4 Design Basis

This section provides the design basis for the proposed SNF WWTP. The design basis was developed based on the existing Niagara Falls WWTP historical data and consider various factors with respect to:

- Population projections
- Projected raw wastewater flows and loadings
- Effluent criteria

4.1 Population Projections

The population projections within the South Niagara Falls area will form the basis of establishing projected wastewater loading and ultimate sizing of the proposed SNF WWTP. The total equivalent population (i.e. sum of residential and equivalent employment population) that can be served by the proposed 30 MLD SNF WWTP was estimated as follows:

- Current Base Population (approximately 15 MLD): estimated based on historical per capita flow of 285 L/cap/d (refer to Section 2.2).
- New Growth Population (approximately 15 MLD): estimated based on per capita flow of 275 L/cap/d as recommended in the 2017 Master Plan (GM BluePlan).

Given the Region's continued efforts to reduce extraneous (infiltration & Inflow) flows to the sanitary collection system and the likelihood that much of the population growth in the South Niagara Falls area will be associated with the installation of new infrastructure (sewers), utilizing the 2017 Master Plan recommended per capita sewage generation rate of 275 L/cap/d is considered appropriate for future growth.

Table 7 summarizes the total serviced population for the new 30 ML/d SNF WWTP.

Table 7 SNF WWTP Projected Equivalent Service Population (at Design Flow of 30 MLD)

Parameter	Total Equivalent Service Population (approx.)	Per Capital Flow (L/cap/d)
Current Base (15 MLD)	53,000 (1)	285
New Growth (15 MLD)	55,000 (2)	275
Overall Projected (30 MLD)	108,000	280 (3)

Notes:
(1) Based on historical per capita flow of 285 L/cap/d.
(2) Based on design per capita flow of 275 L/cap/d for total equivalent service population estimate as recommended in Master Plan (GM Blue Plan, 2017).
(3) Estimated based on design average day flow of 30 MLD and projected population of 108,000.

4.2 Raw Wastewater Flows

The proposed SNF WWTP will be designed to accommodate a raw wastewater flow of 30 MLD. The projected peak flows (i.e. MDF, PHF and PIF) were calculated based on the historical (base) peak flows at the existing Niagara Falls WWTP, plus an allowance for new growth with more typical I/I associated with a separate sewer system.

Table 8 presents the overall projected flows to the proposed SNF WWTP.

Table 8 Design Raw Wastewater Flow

Parameter	Average Day Flow (MLD)	Peak Factor (1)			Peak Flow (MLD)		
		MDF	PHF	PIF	MDF	PHF	PIF
Existing Service Area	15	3.1	4.1	4.5	46	61	67
New Growth	15	2.0	3.0	3.5	30	45	53
Overall	30	2.5	3.5	4.0	76	106	120

Note:
(1) Peak factors for existing service area based on the Niagara Falls WWTP historical data for a combined sewer system; Peak factors for new growth based on typical peaking factors for a separate sewer system.

4.3 Raw Wastewater Characteristics and Loadings

As discussed in Section 2, the design of the proposed SNF WWTP will be based on the combined characteristics and loadings of influent raw wastewater, centrate from the Garner Road Biosolids Facility and hauled waste. This is to accommodate the incremental hydraulic, solids, and organic and nutrient load imposed from the external recycle stream on the plant. The loadings from internal recycle streams will be considered as part of the overall conceptual design for the new plant based on mass balance once the overall treatment train is established.

Raw wastewater loading projections were based on the current base raw wastewater loadings, plus an allowance for new growth. As discussed in Section 2.3, the historical per capita loadings were slightly lower than typical for all parameters. To provide a more conservative design basis for a new facility, typical design per capita loading values were used to develop loadings at the new plant.

Projected centrate flow from the Garner Road Biosolids Facility is 1.73 MLD. The centrate flow was estimated based on pro-rating the current average centrate flow of 1.05 MLD generated from two centrifuges to include a third centrifuge at Garner Road for future growth. The centrate concentrations were based on typical values for a mesophilic anaerobic digester as presented in Table 5 above. A daily average hauled waste flow of 0.15 MLD was assumed with typical values summarized in Table 6 above.

Table 9 presents a summary of design influent wastewater loadings of the combined flows. Design flows and loadings cacluation can be found in Appendix A.

Table 9 Design Influent Wastewater Loadings (at Design Flow of 30 MLD)

Design Parameter	Per Capita Loading (1)	Raw Sewage	Centrate from Garner Road Facility (2)	Hauled Waste (3)	Combined
Average Loadings (kg/d) (4)					
BOD_5	75 g/cap.d	8,100	1,300	1,050	10,450
TSS	90 g/cap.d	9,720	2,600	2,250	14,570
TP	2 g/cap.d	220	90	40	350
TKN	13 g/cap.d	1,400	1,380	105	2,885
Peak Month Loadings (kg/d) (5)					
BOD_5	Peaking factor: 1.3	10,530	1,690	1,370	13,600
TSS		12,640	3,380	2,930	19,000
TP		290	120	50	500
TKN		1,800	1,800	140	3,800

Notes:
(1) Based on Metcalf & Eddy (2003).
(2) Centrate average loadings based on typical concentrations (Metcalf & Eddy, 2003) and a centrate flow of 1.73 MLD.
(3) Based on average hauled waste flow of 0.15 MLD
(4) Based on typical per capita load and a projected total equivalent population of 108,000 (refer to Table 7).
(5) Based on a typical loading factor of 1.3.

The results outlined in the Table 9 indicate that the external sidestream centrate addition will have minor impacts on the plant flow, but have siginficant impacts on the plant loadings, specifically TKN. The design centrate flow is 1.73 MLD, represents 4% of the 30 MLD design flow. The TKN loading loading of raw sewage is doubled due to the centrate addition. The increased loading from the external sidestream centrate addition will have an impact on the unit process selection, sizing and energy consumption of the liquid treatment train at the plant. This will be further addressed in TM No. 2 – Technology Review.

In addition to the raw wastewater characteristics listed in Table 9, other important raw wastewater characteristics include temperature, pH and alkalinity. These are important considerations for secondary treatment design and nitrification.

Table 10 presents a summary of the design values of these raw wasewater characteristics developed based on existing Niagara Falls WWTP historical data (2015 to 2018). A slightly higher minimum winter temperature is proposed for the new WWTP, consistent with reduced inflow/infiltration associated with the new growth.

Table 10 Design Raw Wastewater Characteristics for Nitrification

Parameters	Existing Niagara Falls WWTP Range	Proposed Design Basis (1)
Temperature (°C) (2)	Winter: 7-14 Summer: 13-25	Winter Minimum: 10 Summer Minimum: 13
pH	7.2-7.6	7.4 (7.2-7.6)
Alkalinity (mg/L as $CaCO_3$)	50 – 300 (3)	100

Notes:
(1) Developed based on historical data (2015 to 2018).
(2) Historical inf. temperature data no available. Based on historical effluent temperature data (2015 to 2018).
(3) Data at existing WWTP not available. Based on Niagara Region's drinking water monitoring data.

4.4 Effluent Criteria

As part of the Class EA process, an Assimilative Capacity Study (ACS) has recently completed to develop effluent criteria of the proposed South Niagara Falls WWTP which will discharge to Chippewa Creek. The recommended effluent criteria are presented in Table 11, and an ACS report can be found in Appendix B (Golder, 2020). The plant will have requirements for year-round nitrification; and will have no requirements for tertiary phosphorus removal.

Table 11 Design Effluent Objectives and Limits for the Proposed South Niagara Falls WWTP

Parameters	Effluent Objectives (mg/L) (1)	Effluent Limits (mg/L) (1)
Carbonaceous Biochemical Oxygen Demand ($CBOD_5$)	15	25
TSS	15	25
TP	0.5	0.75
Total Ammonia Nitrogen (TAN) May to October November to April	 6.5 12.0	 8.8 15.0
E. Coli (CFU/ 100 mL) (2)	200	200

Notes:
(1) Based on monthly average concentrations.
(2) Based on monthly geometric mean.

5 Summary

The existing Niagara Falls WWTP historical flow, loading and performance data from 2017 to 2020, along with the Garner Road Biosolids Facility historical centrate and typical hauled waste data, were reviewed and statistically analyzed to develop the design basis for the proposed South Niagara Falls WWTP.

The recommended design basis for the combined raw wastewater, centrate from the Garner Road Facility and hauled waste is summarized in Table 12. The design basis will be used to develop and evaluate alternative design concepts as part of the Phases 3 and 4 of the South Niagara Falls WWTP Class EA Study.

Table 12 Recommended Design Basis for the Proposed South Niagara Falls WWTP

Parameter	Value	Basis
Raw Wastewater Flow		Stage 1 Rated Capacity
Average Day	30 MLD	99.5 percentile historic
Maximum Day	76 MLD	99.5 percentile historic
Peak Hour	106 MLD	Design basis for Plant Inlet
Peak Instantaneous	120 MLD	
Influent Average Concentration (mg/L)		Average load divided by the average flow
BOD_5	330	
TSS	460	
TP	11	
TKN	90	
Influent Average Loading (kg/d)		Typical per capital load (Metcalf & Eddy, 2003), plus centrate loadings from Garner Road, and hauled waste
BOD_5	10,450	
TSS	14,570	
TP	350	
TKN	2,885	
Influent Peak Month Loading (kg/d)		Typical peak month loading factor of 1.3
BOD_5	13,600	
TSS	19,000	
TP	500	
TKN	3,800	
Effluent Objective		2020 ACS (Golder)
$CBOD_5$	15 mg/L	
TSS	15 mg/L	
TP	0.5 mg/L	
TAN (May to November)	6.5 mg/L	
TAN (December to April)	12.0 mg/L	
Effluent Limit		2020 ACS (Golder)
$CBOD_5$	25 mg/L	
TSS	25 mg/L	
TP	0,5 mg/L	
TAN (May to November)	8.8 mg/L	
TAN (December to April)	15.0 mg/L	

6 References

- GM Blue Plan (2017). Niagara Region 2017 Water and Wastewater Master Servicing Plan Update.
- GM Blue Plan (May 2020). Planning Projection Technical Memorandum, South Niagara Falls Wastewater Solutions Schedule C Class Environmental Assessment.
- Metcalf & Eddy, Inc. (2003), Wastewater Engineering: Treatment and Re-Use 4th Ed.", New York, New York, U.S.A.
- Ministry of the Environment, Conservation and Parks (MECP) (2008), Design Guidelines for Sewage Works, Toronto, Ontario, Canada.
- Water Environmental Federation (WEF, 2017). Design of Water Resources Recovery Facilities MOP 8, Fifth Edition.

A

Appendix A

Design Flow and Loading Calculations

Design Flow and Loading Calculations
TM No.1 - Design Basis
South Niagara Falls WWPT Class EA and Conceptual Design, Niagara Region

Calculated by: AF
Updated by: MY
Checked by: TB
Date: Feb 2022

Proposed South Niagara Falls WWTP Design Flows

Parameter	Average Day Flow (MLD)	Peak Factor (1)			Peak Flow (MLD)		
		MDF	PHF	PIF	MDF	PHF	PIF
Existing Service Area	15	3.1	4.1	4.5	47	62	68
New Growth	15	2.0	3.0	3.5	30	45	53
Overall	30	2.6	3.6	4.0	77	107	120

Note: (1) Peak factors for existing service area based on historical data for a combined sewer system; Peak factors for new growth based on typical peaking factors for a separate sewer system.

Proposed South Niagara Falls WWTP Loading Projections

Parameter	Tpyical Design Per Capita Load (g/cap/d) (1)	Raw Sewage Avg. Load (kg/d) (3)	Garner Road Centrate Avg. Load (kg/d) (4)	Hauled Waste Avg. Load (kg/d) (5)	Combined Avg. Load (kg/d)	Raw Sewage Peak Month Load (kg/d) (7)	Garner Road Centrate Peak Month Load (kg/d) (7)	Hauled Waste Peak Month Load (kg/d) (7)	Combined Peak Month Load (kg/d) (7)	Raw Sewage Conc. (mg/L)	Typical Centrate Conc. (mg/L) (1)	Typical Hauled Waste Conc. (mg/L) (2)	Combined Avg. Conc. (mg/L) (6)
BOD_5	75	8,100	1,300	1,050	10,450	10,530	1,690	1,370	13,600	270	750	7,000	330
TSS	90	9,720	2,600	2,250	14,570	12,640	3,380	2,930	19,000	324	1,500	15,000	460
TP	2.0	220	90	40	350	290	120	50	500	7	50	250	11
TKN	13.0	1,400	1,380	105	2,885	1,820	1,790	140	3,800	47	800	700	90

Notes:
(1) Metcalf & Eddy (2003).
(2) Table 3-8 US EPA Handbook - Septage Treatment and Disposal, 1984
(3) Based on Population Projections

Existing Service Area Equivalent Population	53,000	person
New Growth Equivalent Population	55,000	person
Total Equivalent Population	108,000	person
(4) Based on Projected Centrate flow	1.73	MLD
(5) Based on Projected Hauled Waste flow	0.15	MLD
(6) Based on Combined avg. Load and Flow	31.88	MLD
(7) Based on Typical Peak Loading Factor	1.3	

B

Appendix B

Assimilative Capacity Study Report

REPORT

South Niagara Falls Wastewater Solutions Schedule C Class Environmental Assessment

Detailed Assimilative Capacity Study

Submitted to:

Niagara Region

1815 Sir Isaac Brock Way

Thorold ON L2V 4T7

Submitted by:

Golder Associates Ltd.

6925 Century Avenue, Suite #100, Mississauga, Ontario, L5N 7K2, Canada

+1 905 567 4444

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ACRONYMS AND ABBREVIATIONS

Acronym or Abbreviation	Description
ACS	Assimilative Capacity Study
BOD_5	Biochemical Oxygen Demand
$CBOD_5$	Carbonaceous Biochemical Oxygen Demand
CCME	Canadian Council of Ministers of the Environment
CSO	Combined Sewer Overflow
E. coli	*Escherichia coli*
EA	Environmental Assessment
ECA	Environmental Compliance Approval
GS	Generating Station
HEPC	Hydro Electric Power Canal
ICD	International Control Dam
INCW	International Niagara Control Works
MECP	Ministry of the Environment, Conservation and Parks
MOEE	Ministry of Energy and Environment
NOAA	National Oceanic and Atmospheric Administration
NPCA	Niagara Peninsula Conservation Authority
NYPA	New York Power Authority
OPG	Ontario Power Generation
the Project	South Niagara Falls Wastewater Solutions Schedule C Class EA
PWQMN	Provincial Water Quality Monitoring Network
PWQO	Provincial Water Quality Objectives
SAB	Sir Adam Beck GS
SLSMC	St. Lawrence Seaway Management Corporation
TSS	Total Suspended Solids
USGS	United States Geological Survey
WSC	Water Survey of Canada
WWTP	Wastewater Treatment Plant

UNITS OF MEASURE

Symbol or Unit	Description
cfs	Cubic feet per second
cfu	Colony-forming unit
kg/d	kilograms per day
km	kilometre
km^2	Square kilometres
m	metre
µg/L	Microgram per litre
mg/L	Milligrams per litre
MLD	Megalitres per day
m^3/s	Cubic metres per second
mL	Millilitre
ºC	Degrees Celsius
%	Percent

1.0 INTRODUCTION

The Regional Municipality of Niagara (Niagara Region) is currently conducting a Schedule “C” Municipal Class Environmental Assessment (EA) for a proposed Wastewater Treatment Plant (WWTP) in the southern area of the City of Niagara Falls. As well as providing other ancillary services, Golder Associates Ltd. (Golder) has been retained to conduct an Assimilative Capacity Study (ACS) in support of the South Niagara Falls Wastewater Solutions Schedule C Class EA Project (the Project), which is the subject of this technical report.

1.1 Study Background

With significant future regional growth and urban intensification forecast for the area, the 2017 Niagara Region Master Servicing Plan provided a long-term wastewater solutions strategy to improve the existing collection system and add a new, second wastewater treatment facility in South Niagara Falls that can accommodate phased growth, provide wastewater service to currently subserviced areas, reduce pressure on existing wastewater infrastructure, decrease the magnitude and frequency of untreated combined sewer overflows and WWTP bypasses and, in doing so, enhance overall environmental performance.

Wastewater collection within Niagara Falls is currently facilitated through a number of collection systems and pumping stations. These systems convey the wastewater to the existing Niagara Falls WWTP (sometimes referred to as the Stanley Avenue WWTP). Many of the components of the collection system are nearing their design capacity.

The 2017 Master Servicing Plan identified several candidate discharge locations for a new WWTP in South Niagara Falls that could potentially accept an effluent discharge rate of up to 30 Megalitres per day (30 MLD). The preferred location was discharge from the south bank into Chippewa Creek approximately 350 m east of Triangle Island and chosen based on available property for the new WWTP, existing and required infrastructure to convey raw sewage to the new plant and a screening level assimilative capacity assessment (see Appendix A). The preferred discharge location is identified as Location 3 on Figure 1. Details of the selection process were presented at several Public Information Centres (PICs) and will be fully documented in the Environmental Study Report.

1.1.1 Study Area Overview and Nomenclature

The hydrology of the study area has been highly modified and regulated from the natural predevelopment conditions that existed prior to the 1950s. During the 1950s, the Hydro Electric Power Canal (HEPC) was constructed from the Welland River (upstream of Horseshoe Falls) to the Sir Adam Beck Generating Station (GS) which discharges to Niagara Gorge. As a result, the flow within last 6.5 km of the Welland River was reversed to direct a portion of Niagara River flows towards the HEPC. The section from the Niagara River to Triangle Island is referred as Chippewa Creek. The amount of flow that is diverted is primarily determined by the following factors:

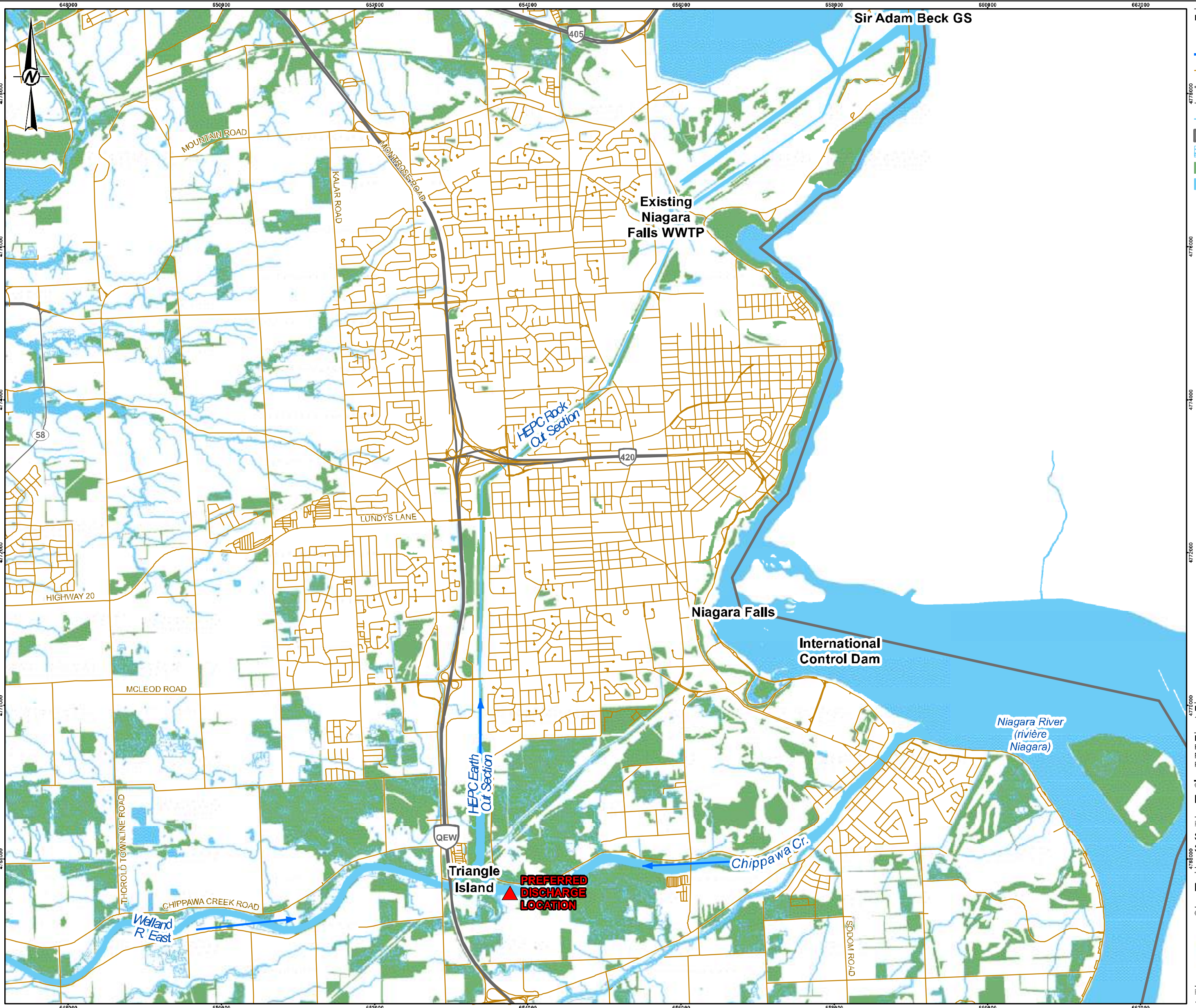
- the operation of the International Control Dam (ICD) in the Niagara River; which can alternatively increase or decrease the water level in the Niagara River at the mouth of Chippewa Creek; and
- upstream flows in the Niagara River which are determined by water levels at the outlet of Lake Erie, that are influenced by both long-term weather patterns and short-term meteorological events (such as seiching).

The daily operation of the ICD is influenced by the electrical demands and markets in both Ontario and New York State as well as maintaining minimum flow over the falls during tourist periods.

In addition, construction of the Welland Canal to the west of the study area has modified the hydrology and drainage area of the Welland River and several small contributing tributaries. The Welland River passes under the Welland Canal at two locations via siphons that may alter the flow in the river. The Lyons Creek watershed area was also decreased by the Welland Canal to the extent that water must now be pumped from the Welland Canal into Lyons Creek to maintain a minimum flow requirement.

For the purposes of maintaining consistent terminology, key surface water features referred to in this ACS use a naming convention adopted by the Ministry of the Environment, Conservation and Parks (MECP), the Niagara Peninsula Conservation Authority (NPCA), and Ontario Power Generation (OPG). Specifically, these key surface water features include:

- **International Control Dam (ICD):** This multi-gated dam in the Niagara River built in 1954 is located approximately 800 m above the Horseshoe Falls and is used to control flows to the Sir Adam Beck GS operated by OPG, the Robert Moses GS operated by the New York Power Authority (NYPA) and the American Falls operated according to Niagara River Treaty (1950). In other literature and documentation, the ICD has sometimes also been referred to as the International Niagara Control Works (INCW).
- **Chippewa–Grass Island Pool:** This is the area of the Niagara River upstream of the ICD where water levels vary with upstream flow and the operation of the ICD.
- **Hydro Electric Power Canal (HEPC):** This is a canal that conveys diverted flow from the Niagara River (via Chippewa Creek) to the Sir Adam Beck GS.
- **Chippewa Creek:** This is a former portion of the Welland River that flows from the Niagara River to the HEPC when the HEPC is in operation (e.g., reverse flow to natural conditions). During the construction of the HEPC, the width and depth of this section of river were increased to accommodate the increased flow.
- **Triangle Island:** this is a small, constructed island at the junction of the Welland River East, Chippewa Creek, and the HEPC. During normal operation of the HEPC, the diverted flow from the Niagara River flows past the northeast side of Triangle Island from Chippewa Creek into the HEPC while flow from the Welland River East flows past the northwest side of Triangle Island into the HEPC. The channel to the south of Triangle Island is narrower and shallower than the other channels and does not typically have significant flows. Triangle Island is also the location of the safety booms (northeast and northwest sides) used to prevent boat traffic from entering the HEPC.
- **Earth Cut Section:** This is the wide portion of the HEPC dug into soil between Triangle Island and the Rock Cut Section of the HEPC and is approximately 1.5 km long.
- **Rock Cut Section:** This is the narrower and deeper section of the HEPC cut into bedrock below the Earth Cut Section. The rock cut section of the HEPC is approximately 12 km long and ends at the Sir Adam Beck GS.
- **Welland River East:** This is the portion of the Welland River upstream of triangle island. MECP / NPCA use this convention to distinguish the sections of the Welland River east or west of the Welland Canal.



LEGEND

- PREFERRED DISCHARGE LOCATION
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- WATERCOURSE
- INTERNATIONAL BORDER
- WETLAND
- WOODED AREA
- WATERBODY

0 1 2
1:50,000 KILOMETRES

NOTE(S)
1. ALL LOCATIONS ARE APPROXIMATE.

REFERENCE(S)
BASE DATA COURTESY OF MNRF LIO. PRODUCED BY GOLDER ASSOCIATES UNDER LICENSE FROM ONTARIO MINISTRY OF NATURAL RESOURCES AND FORESTRY
PROJECTION: UTM ZONE 17N DATUM: NAD 83

CLIENT
REGIONAL MUNICIPALITY OF NIAGARA

PROJECT
SOUTH NIAGARA FALLS WASTEWATER SOLUTIONS
SCHEDULE C CLASS ENVIRONMENTAL ASSESSMENT

TITLE
LOCATION OF PROJECT AREA

CONSULTANT: GOLDER

YYYY-MM-DD	2020-05-19
DESIGNED	PR
PREPARED	PR
REVIEWED	GVA
APPROVED	GVA

PROJECT NO.	CONTROL	REV.	MAP
18104462	0010	A	1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM ANSI B

1.1.2 Selected Discharge Location

The preferred discharge location is located on the south bank of Chippewa Creek approximately 350 m east of Triangle Island. The effluent from the new WWTP would mix with flow that is composed mainly by water from the Niagara River diverted into the HEPC and minimal flow from Lyons Creek.

The creek channel in the area of the outfall is effectively a constructed channel with a uniform width, depth and side slopes that follows the original path of the Welland River prior to the construction of the HEPC. The channel is approximately 100 m wide and 12 m deep with approximately 1:2 (horizontal:vertical) side slopes. During typical operation of the ICD, the flow in the creek is from east to west and the current speeds range from approximately 0.35 to 0.5 m/s with an average of approximately 0.42 m/s.

1.2 Study Purpose

The purpose of this ACS is to provide an assessment of the preferred discharge location (Chippewa Creek) in support of the Municipal Class EA by:

1) Evaluating the assimilative capacity of the discharge location, considering the monthly characteristics of key water quality parameters that could be affected by treated effluent discharge.
2) Determining the environmental constraints of the discharge location with respect to assimilating a treated wastewater discharge of 30 MLD.
3) Identifying the discharge concentration limits of key water quality parameters to meet Provincial Water Quality Objectives (PWQOs), to meet Canadian Council for Ministers of the Environment (CCME) criteria (where PWQOs are not available), or to maintain water quality in accordance with MECP Policy 2 requirements at the discharge location.
4) Developing a conceptual outfall design and evaluating the performance of the outfall in terms of effluent mixing with the receiving water.

1.3 General Study Approach and Report Outline

The characterisation of the discharge location considered in this study is based on available sources discussed in Appendix A. The structure of this detailed ACS report for Chippewa Creek is presented in the following order:

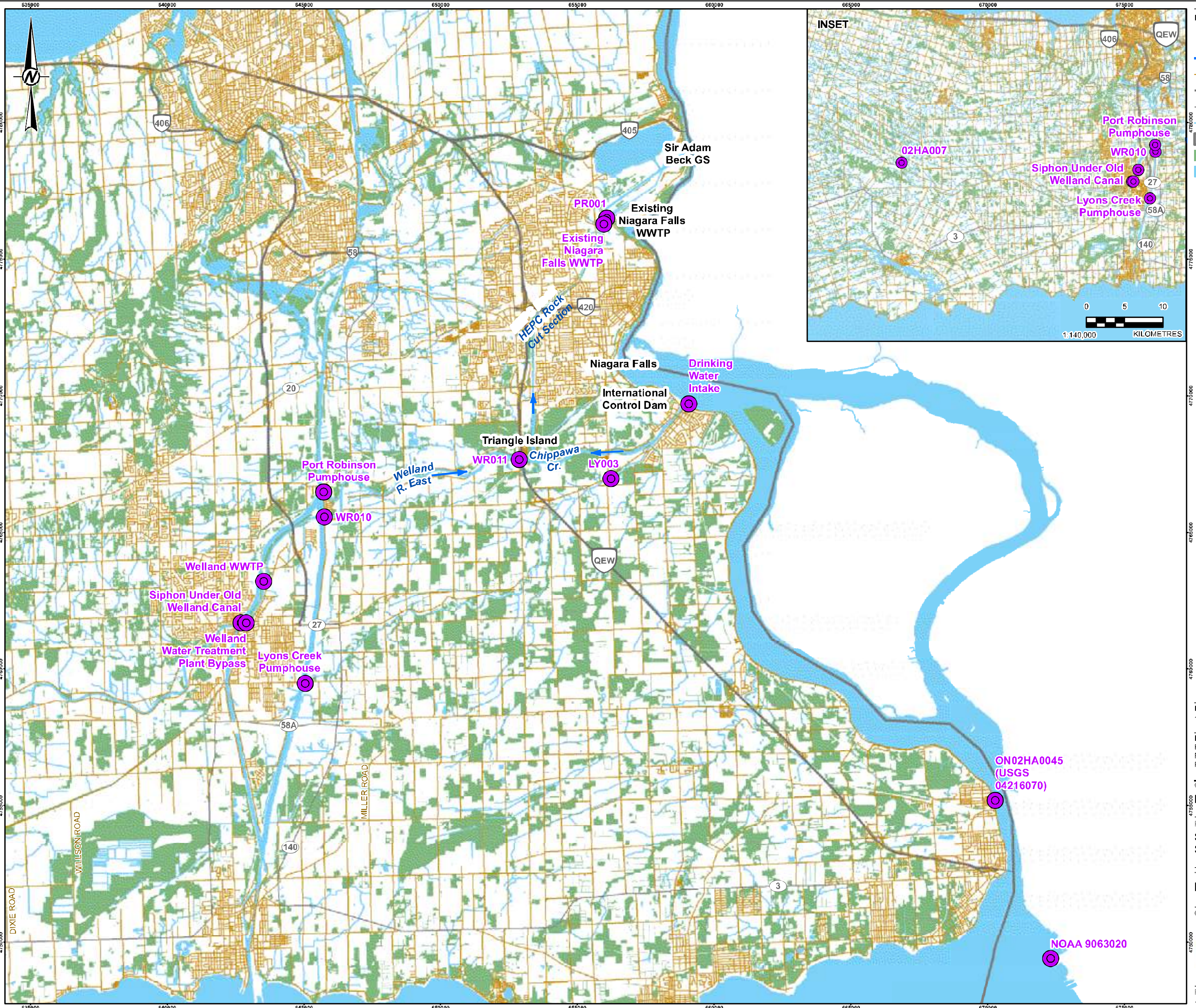
- Section 2 details the background information obtained and used to characterise monthly water quality and flow conditions in the study area.
- The hydrological nature of the selected location required a slightly modified approach compared to conventional Assimilative Capacity Studies. Namely, the flow in Chippewa Creek and the HEPC is heavily regulated, which meant that the conventional 7Q20 approach to flow derivation was replaced with a stochastic approach. Section 3 introduces the modelling approach adopted and identifies relevant monthly and/or environmental constraints, as well as identifying the maximum allowable effluent concentrations at each discharge location to achieve regulatory compliance. Section 3 also includes the mixing zone assessment and the evaluation of the expected performance of the proposed outfall conceptual design.
- Based on the constraints identified in Section 3, Section 4 recommends effluent limits for each parameter as well as the predicted parameters in the effluent plume immediately downstream of the outfall and the expected effect of the Project on water quality at selected downstream locations.

- Section 5 summarises the key conclusions and recommendations of the detailed ACS for the preferred discharge location into Chippewa Creek.

This study assesses the assimilative capacity and water quality effects at two compliance points for each discharge option. The local compliance point is located immediately downstream of the discharge. In order to consider the cumulative effects of existing discharges to the HEPC, the system compliance point is located in the HEPC immediately downstream of the existing Niagara Falls WWTP and upstream of the confluence with the power tunnels.

2.0 BACKGROUND INFORMATION AND DATA REVIEW

This section provides details and summaries of the data used in the ACS. The location of the monitoring locations where the data were collected are shown on Figure 2.



LEGEND

- REGIONAL SAMPLING STATION LOCATION
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- WATERCOURSE
- INTERNATIONAL BORDER
- WOODED AREA
- WATERBODY

0 5 10
1:140,000 KILOMETRES

NOTE(S)
1. ALL LOCATIONS ARE APPROXIMATE.

REFERENCE(S)
BASE DATA COURTESY OF MNRF LIO, PRODUCED BY GOLDER ASSOCIATES UNDER LICENSE FROM ONTARIO MINISTRY OF NATURAL RESOURCES AND FORESTRY
PROJECTION: UTM ZONE 17N DATUM: NAD 83

CLIENT
REGIONAL MUNICIPALITY OF NIAGARA

PROJECT
SOUTH NIAGARA FALLS WASTEWATER SOLUTIONS
SCHEDULE C CLASS ENVIRONMENTAL ASSESSMENT

TITLE
DATA COLLECTION AND MONITORING LOCATIONS

CONSULTANT
GOLDER

YYYY-MM-DD	2020-05-19
DESIGNED	MM/PR
PREPARED	PR
REVIEWED	GVA
APPROVED	GVA

PROJECT NO.	CONTROL	REV.	MAP
18104462	0007	A	2

2.1 Flow Data

2.1.1 Water Management in Study Area

The flow in Chippewa Creek and the HEPC has been controlled since 1921. The ICD has been in operation since 1954 and is jointly funded and controlled by OPG and NYPA in accordance with the 1950 Niagara Treaty (Canada, 1950) and a Memorandum of Understanding between the two power companies which are intended to maximize the beneficial use of the hydro electric potential of the Niagara River, while maintaining the scenic value of Niagara Falls for tourism and other uses of water in the Niagara River. The treaty stipulates that:

- Scenic flow is allocated first, domestic use second, navigational requirements third, and power generation fourth.
- Any river flow diverted for hydro electric power is to be split equally between both countries.
- During tourist times, the flow over the falls must be at least 2,832 m^3/s (100,000 cfs). Tourist times are defined as 8 AM to 10 PM from April 1 to September 15 and 8 AM to 8 PM from September 16 to October 31.
- The specified minimum flow over the falls is at least 1,416 m^3/s (50,000 cfs) at all other times.
- If the upstream flow in the Niagara River is less than the specified minimum flows, no river flow is to be diverted to the power canals.

Water levels in the Chippewa-Grassy Island Pool are regulated in accordance with the 1993 Directive of the International Niagara Board of Control.

In addition, OPG is required to maintain a minimum flow of 240 m^3/s to the HEPC via Chippewa Creek to ensure that water from the Niagara River reaches the existing drinking water intake of the City of Niagara Falls Water supply plant located near the junction of Chippewa Creek and the Niagara River (Kowalski 2019). Niagara Region is currently in the process of relocating the water supply intake to the Niagara River upstream of Chippewa Creek.

2.1.2 Welland River East

In general, low flow frequency analysis of natural flows is used to generate the low-flow conditions (7Q20) to assess the assimilative capacity of the receiving water body (MOE 1994a). The Welland River East, however, is a complex hydrologic system characterized by natural flows and supplemental flows and the low-flow conditions are dominated by the supplemental flows. As a result, the 7Q20 would not be applicable for this specific assessment. Previous Assimilative Capacity Studies in the Welland River East have successfully applied an approach where the low flows conditions are based on combination of natural and supplemental flows rather than an approach based solely on the 7Q20, as shown in the ACS completed for the Welland Wastewater Treatment Plant (XCG 2007).

2.1.2.1 Natural Flows in the Welland River East

Regional station data was used to estimate natural flow for the Welland River East. Flow data for the Welland River below Caistor Corners (station 02HA007) from the WSC are available from 1957 to 2017. Flows at the site are calculated based on the prorated watershed area of the site (906 km^2) and the total watershed area of the gauged station (223 km^2). Natural flows in the system are generally low with punctual peak flows recorded during storm events and snowmelt.

Since supplemental flows are significantly higher than average natural flows in the system (i.e., approximately double the annual average flows), natural flows in the Welland River East become relevant only under peak flow conditions. Therefore, flows were prorated between the gauging station (223 km^2) and the area at the site (906 km^2) according to the Transposition of Flood Discharges Method (MTO 1997) applying a coefficient of 0.75 to represent peak flows (the coefficient used for average and low flows is 1.0).

The estimated natural flows yield an average annual flow of 6.50 m^3/s with estimated maximum and minimum flows in the range of 132.41 m^3/s and 0.046 m^3/s. The 7Q20 for the natural flows based on the Log Pearson Type III distribution would yield 0.004 m^3/s.

2.1.2.2 Supplemental Flow from Welland Canal into Welland River East

Supplemental flows enter the Welland River East from the Welland Canal (St. Lawrence Seaway Management Corporation [SLSMC] 2019) as follows:

- A series of ports in the roof of the old syphon provide flow from the canal into the river. Depending on the season and water levels in the canal, the total flow ranges from 5 to 7 m^3/s.
- The bypass of the Welland Water Treatment Plant provides a flow between the canal and the river that ranges from 4 m^3/s to 6 m^3/s.
- A pump at Port Robinson provides a flow of 0.97 m^3/s to a side channel of the Welland River East, which was cut-off from the main branch of the river during the straightening of the canal in the 1950s.
- The effluent from the Welland Wastewater Treatment Plant provides a flow of 0.8 m^3/s (XCG 2007).

In general, the supplemental flows from the Welland Canal are from Lake Erie and have better water quality than that of the upstream areas of the Welland River.

Monthly estimates of the supplemental flows for the siphon ports, Port Robinson Pump, the Welland Water Treatment Plant, and the Welland WWTP were provided by the SLSMC (SLSMC 2019) for the period 2014 to 2019 and are summarized in Table 1.

Table 1: Summary of Supplemental Flows from Welland Canal into the Welland River East

Source	Old Welland Canal at Old Siphon[1]		Welland Water Treatment Plant[1]		Port Robinson Pump[1]	Welland WWTP[2]
	Minimum (m^3/s)	Average (m^3/s)	Minimum (m^3/s)	Average (m^3/s)	Average (m^3/s)	Average (m^3/s)
January	5.58	5.94	4.80	5.17	0.97	0.80
February	5.17	5.61	4.45	4.87		
March	5.85	6.35	5.03	5.48		
April	6.54	6.88	5.62	5.94		
May	6.03	6.60	5.19	5.77		
June	6.69	6.86	5.75	5.88		
July	6.82	6.90	5.87	5.90		
August	6.68	6.85	5.75	5.89		
September	6.62	6.81	5.69	5.86		
October	6.56	6.79	5.64	5.84		
November	6.87	7.04	5.90	6.06		
December	7.03	7.09	6.04	6.10		

Notes:

1. SLSMC 2019.
2. XCG 2007.

2.1.3 Lyons Creek

During the construction of the Welland Canal, the watershed of Lyons Creek (originally draining to Chippewa Creek) was split between the western section which now drains into the Welland Canal, and the eastern section which still drains into Chippewa Creek. As a result of this reduction in drainage area, the natural flows in Lyons Creek are supplemented by the pumping of water from the Welland Canal at the location where the main channel of Lyons Creek was interrupted to the eastern section of Lyons Creek, which is of interest for this study.

- Flow data for Lyons Creek is not available. Natural flows were estimated using Regional station data for the Welland River Below Castor Corners (station 02HA007) from the WSC, by prorating the watershed area for the site (88 km^2) and the total watershed area of the gauged station (223 km^2).
- Supplemental flows vary seasonally ranging from 0.142 m^3/s between December to March (when Welland Canal is drained) to 0.283 m^3/s during the rest of the year (SLSMC 2019).

2.1.4 Hydro Electric Power Canal (HEPC)

Flow from the Niagara River is diverted to the Sir Adam Beck GS from the Chippewa-Grass Island Pool via three tunnels and the HEPC. Under normal operating conditions, each of these conveyances carries approximately one quarter of the total diverted flow. The flow in the HEPC and tunnels can vary hourly and seasonally due to flow variations in the Niagara River, minimum flow requirements over the falls (see Section 2.4.1), electrical demand, and the market price for electricity.

The flow data provided by OPG (Kowalski 2019) represents the total flow diverted by OPG from the Niagara River to the HEPC and the three tunnels. Typically, the flow in the HEPC represents 27% of the total diverted flow.

Hourly flow data provided by OPG for a three-year period (2016 to 2018) was used as a basis for the following observations regarding the flow in the HEPC:

- The hourly flow rate ranged from 292 m^3/s to 624 m^3/s with an average of 429 m^3/s.
- Flow rates are typically highest during the summer months (446 m^3/s) and lowest in the fall (411 m^3/s).

Typically, the flows are lowest at 4:00 AM (402 m^3/s) and highest at 6:00 PM (456 m^3/s).

2.1.5 Chippewa Creek

Water from the Niagara River is diverted into Chippewa Creek based on the water levels in the Chippewa-Grass Island Pool. Chippewa Creek extends approximately 6.5 km from the Niagara River to Triangle Island. Lyons Creek drains to the south shore of Chippewa Creek approximately 2 km west of the Niagara River.

Given the highly regulated system, flow in Chippewa Creek was estimated in the model based on the flow demand in the HEPC and the estimated flows contributing to the system from the Welland River East and Lyons Creek. The estimated flow (diverted from Niagara River) was calculated in the modelling exercise.

2.1.6 Existing Niagara Falls Wastewater Treatment Plant

The existing Niagara Falls WWTP operates at an average flow of approximately 0.472 m^3/s (40,810 $m^{3/}$day). For the ACS modelling, the effluent flow was maintained at the existing rated capacity of 0.79 m^3/s (68,300 m^3/d). The effluent from the plant to the HEPC and immediately upstream from the system compliance point (upstream of Sir Adam Beck GS).

2.1.7 Combined Sewer Overflows (CSOs) and Wastewater Treatment Plan Bypass

Niagara Region has a total of five Regional CSOs discharging into the HEPC from regional pumping stations. Discharges from the CSOs into the HEPC are primarily triggered by storm events. Since the ACS focuses on dry events, CSOs were excluded from the detailed analysis for the proposed discharge location to Chippewa Creek.

2.2 Water Quality Data

For the ACS, the parameters of concern include total ammonia, unionized ammonia, nitrate, phosphorus, *Escherichia coli* (*E. coli*), dissolved oxygen, Carbonaceous Biochemical Oxygen Demand ($CBOD_5$), and total suspended solids (TSS). The assessment used pH and water temperature in the Niagara River to estimate unionized ammonia concentration using the equations provided by the MECP (Ministry of Energy and Environment [MOEE] 1994).

The monthly data summary (for each flow source) includes the geometric mean and 75th percentile values (or 25th percentile for dissolved oxygen) for all parameters and available water quality monitoring associated with each individual flow source. These percentiles are used in subsequent analysis as follows:

- The 75th percentile values for total ammonia, nitrate, total phosphorus, E. coli, CBOD5 (when sufficient data was available), and TSS were used as the background concentrations when estimating the maximum allowable effluent concentrations.
- The 75th percentile values of pH and water temperature from the Niagara River and HEPC were used to estimate the maximum allowable concentration of total ammonia in the effluent, based on the estimated maximum allowable effluent concentration for unionized ammonia. The most restrictive value was used to estimate the maximum allowable concentration of total ammonia in the effluent.
- If more than one water quality monitoring station was available for any given flow source, the maximum reported 75th percentile value was used for conservatism in the modelling exercise.
- The 25th percentile values for dissolved oxygen were used as the background concentrations when estimating the maximum allowable effluent concentrations for CBOD5.

2.2.1 Applicable Water Quality Guidelines

Applicable PWQOs for the parameters discussed in this memorandum are presented in Table 2 and are discussed in the following points.

- Since the study area is effectively a river, the PWQO for phosphorus for the avoidance of excessive plant growth in rivers and streams (0.03 mg/L) was used.
- Since there is no PWQO for nitrate, the Canadian Council of Ministers of the Environment (CCME) guideline was selected.
- Seasonal temperature and pH values were used to determine the limits for total ammonia based on the PWQO for unionized ammonia.
- Since the Niagara River, Lyons Creek, and Welland River East are all considered warm water aquatic habitat (NPCA 2011), the dissolved oxygen guideline for warm water fisheries was used.
- The PWQO for fecal coliforms (*E. coli*) is for recreational use (e.g., beaches).

- Since the new WWTP is not expected to release a thermal discharge or alter the pH in the receiving waters, water temperature and pH were excluded from the modelling exercise.
- Since there is no PWQO for total suspended solids, the CCME guideline for clear flow (low flow) was selected

Table 2: Summary of Applicable Water Quality Objectives

Parameter	PWQO or CCME Guideline
Unionized Ammonia	0.0164 mg/L as N[1]
Total Ammonia	Estimated from unionized ammonia criteria based on ambient water temperature and pH using equations in the Provincial Water Quality Objectives (MOEE 1994)
Nitrate	3 mg/L as N[2,3]
pH	6.5 to 8.5[1,4]
E. coli	100 cfu/100 mL[1,3]
Total Phosphorus	0.03 mg/L to avoid excessive plant growth in rivers and streams[1]
Dissolved Oxygen	47% of saturation or 4 mg/L above 20ºC for warm water fisheries[1,5]
Total Suspended Solids	During clear flow (low flow): Maximum average increase of 5 mg/L from background levels for longer term exposures (24 hours to 30 days).[2]
Water Temperature	10ºC above background or 30ºC for thermal discharges[1,4]

Notes:

1. Provincial Water Quality Objectives (MOEE 1994).
2. Guideline for freshwater aquatic life in CCME Guidelines (CCME 2014).
3. PWQO for *E. coli* is for recreational use (e.g., swimming beaches).
4. Since the new WWTP is not expected to release a thermal discharge or alter the pH in the receiving waters, water temperature and pH were excluded from the modelling exercise (explicitly) but used to assess capacity in the system for unionized ammonia.
5. Since the Niagara River, Lyons Creek, and Welland River East are all considered warm water aquatic habitat (NPCA 2011), the dissolved oxygen guideline for warm water fisheries was used.

2.2.2 Welland River East

For the water quality assessment of the Welland River East, data from two monitoring stations were used:

- immediately west (upstream) of Triangle Island at Montrose Road (WR011) with available data from 2011 to 2018; and
- further west (upstream), where the Welland River crosses at the Welland Canal (WR010) with available data from 2003 to 2018.

Water quality data for the Welland River East was provided by NPCA. A summary of the monthly water quality geo-mean and 75th percentile values for WR010 and WR011 are presented in Table 3.

The flows in the Welland River East are a combination of supplemental flows from the Welland Canal (which is effectively water from Lake Erie) and natural drainage from the upper sections of the Welland River Watershed. The water from the Welland Canal is typically of better quality than that of the upper Welland River (e.g., lower phosphorus concentrations). The screening level ACS (Appendix A) demonstrated that during high natural flows, total phosphorus concentrations are elevated.

Water quality in the Welland River East consistently exceeds the PWQO guidelines for phosphorus and *E. coli.* Comparing the 75th percentile concentrations for both stations showed that, overall, water quality parameters do not show distinctive trends between upstream (WR010) and downstream (WR011), with maximum monthly values generally alternating between the stations. The highest 75th percentile concentrations are observed, respectively on: March, January, and February, for total ammonia; January, February, and December for Nitrate; January, December, and November for *E. coli*; March, June, and November to January for total phosphorus.

The GoldSim model uses the monthly 75th percentile of ammonia, *E. coli*, nitrate, and total phosphorus. For each parameter, the highest 75th percentile value from WR011 and WR010 was selected. The decision to use this approach is based on the uncertainty of WR011 (as it would be influenced by flow from Niagara River) and the additional sources which could affect water quality in the reach between WR010 and WR011. Using the highest value of the two stations yields a conservative approach for prediction of assimilative capacity of the system. The assimilative capacity of the system for ammonia is based on the regulatory limit of unionized ammonia, ammonia in the system (based on 75th percentile), and 75th percentile values of pH and temperature.

Table 3: Summary of Monthly Water Quality Concentrations for Welland River East

Month	Station	Number of Samples[1]	Total Ammonia (mg/L)		Nitrate (mg/L)		*E. coli* (cfu/100 mL)		Total Phosphorus (mg/L)		Dissolved Oxygen (mg/L)		Total Suspended Solids (mg/L)		Water Temperature (°C)		pH	
			Geo-mean	75th	Geo-mean	75th	Geo-mean	75th	Geo-mean	75th	Geo-mean	25th	Geo-mean	75th	Geo-mean	75th	Geo-mean	75th
January	WR010	1	0.11	0.11[3]	2.29	2.29[3]	--[5]	--[5]	**0.133**	**0.133[3]**	17.3	17.3[3]	--[5]	--[5]	0.97	0.97[3]	7.82	7.82[3]
	WR011	1	0.68	0.68[3]	2.44	2.44[3]	**9000**	**9000[3]**	**0.130**	**0.130[3]**	16.0	16.0[3]	22.0	22[3]	1.16	1.16[3]	7.88	7.88[3]
February	WR010	4	0.24	0.52	1.67	2.36	--[5]	--[5]	**0.079**	**0.079[3]**	12.9	13.1[4]	--[5]	--[5]	2.07	2.10	7.82	8.09
	WR011	1	0.32	0.32[3]	2.21	2.21[3]	**680**	**680[3]**	**0.110**	**0.110[3]**	13.1	13.1[3]	31.0	31[3]	2.26	2.26[3]	7.58	7.58[3]
March	WR010	7	0.13	0.48	1.25	1.42	--[5]	--[5]	**0.109**	**0.200**	13.3	15.2	--[5]	--[5]	1.42	4.82	8.02	8.17
	WR011	3	0.46	0.97	1.22	1.41	**1173**	**2700**	**0.073**	**0.100**	13.2	13.6[4]	7.5	8.9[4]	3.80	5.16[4]	8.02	8.12
April	WR010	13	0.18	0.22	0.76	1.21	--[5]	--[5]	**0.078**	**0.143**	11.9	12.7	--[5]	--[5]	9.13	10.92	8.15	8.23
	WR011	7	0.12	0.27	0.51	0.89	23	**293**	**0.044**	**0.110**	12.2	13.7	7.1	24.0	7.42	11.34	8.08	8.17
May	WR010	14	0.16	0.22	0.61	0.91	--[5]	--[5]	**0.059**	**0.103**	11.3	12.6	--[5]	--[5]	14.74	16.42	8.06	8.28
	WR011	7	0.13	0.29	0.56	0.71	36	**118**	**0.054**	**0.080**	11.3	13.8	24.3	42.5	15.51	17.50	7.86	8.18
June	WR010	13	0.21	0.32	0.48	0.74	--[5]	--[5]	**0.063**	**0.128**	9.73	10.8	--[5]	--[5]	20.81	22.02	8.17	8.28
	WR011	6	0.09	0.18	0.65	1.72	32	**215**	**0.071**	**0.168**	9.66	10.7	13.1	25.5	21.89	24.00	8.00	8.20
July	WR010	12	0.10	0.17	0.33	0.48	--[5]	--[5]	**0.056**	**0.072**	8.93	10.5	--[5]	--[5]	23.34	24.52	8.17	8.27
	WR011	5	0.08	0.10	0.38	0.50	37	**130**	**0.058**	**0.073**	10.7	12.1	7.9	25.1	25.30	26.56	8.17	8.25
August	WR010	13	0.12	0.20	0.21	0.33	--[5]	--[5]	**0.047**	**0.056**	9.19	10.1	--[5]	--[5]	23.72	24.68	8.18	8.26
	WR011	5	0.04	0.07	0.13	0.23	15	**140**	0.018	**0.047**	8.74	9.28	1.7	5.0	24.58	26.65	8.08	8.17
September	WR010	15	0.10	0.20	0.34	0.46	--[5]	--[5]	**0.049**	**0.062**	8.63	9.90	--[5]	--[5]	20.55	22.35	8.18	8.27
	WR011	7	0.08	0.21	0.38	0.48	55	**1673**	0.032	**0.050**	9.54	10.9	3.7	7.0	22.72	25.63	7.95	8.15
October	WR010	14	0.10	0.20	0.48	1.14	--[5]	--[5]	**0.065**	**0.119**	9.91	11.2	--[5]	--[5]	13.62	15.59	8.16	8.20
	WR011	6	0.09	0.15	0.45	1.55	27	**604**	**0.041**	**0.115**	8.99	10.7	6.5	26.8	19.28	23.48	8.04	8.22
November	WR010	12	0.11	0.30	0.91	1.79	--[5]	--[5]	**0.076**	**0.129**	12.5	14.6	--[5]	--[5]	7.94	9.66	8.21	8.30
	WR011	7	0.12	0.23	0.67	0.91	**153**	**2228**	**0.049**	**0.090**	10.0	13.3	9.5	26.5	13.78	14.52	8.08	8.09
December	WR010	0	0.11[2]	0.20[2]	1.60[2]	0.04[2]	--[5]	--[5]	**0.104[2]**	**0.131[2]**	14.9[2]	15.9[2]	--[5]	--[5]	4.46[2]	5.32[2]	8.02[2]	8.06[2]
	WR011	0	0.40[2]	0.46[2]	1.56[2]	1.68[2]	**4577[2]**	**5614[2]**	**0.089[2]**	**0.110[2]**	11.04	14.24	15.7[2]	24.2[2]	10.35	15.31	7.98[2]	7.99[2]

Notes:

1. Total number of samples collected for the period of record for Welland River WR010 (2003 to 2018) and Welland River WR011 (2011 to 2018) per the month of interest
2. Value calculated as average of previous and next month
3. Insufficient samples to develop a distribution. Value corresponds to geo-mean.
4. Insufficient samples to develop a distribution. Value corresponds to maximum monthly value.
5. No data available
6. Highlighted values correspond with input to the GoldSim model.
7. Bold values indicate exceedances of applicable PWQO.

2.2.3 Niagara River

The water quality in the Niagara River was quantified by compiling data from three sources since no one location offered a full complement of data for all required parameters. The data sources were:

- the Niagara River at Fort Erie (ON02HA0045) from 1981 to 1999 (total phosphorus, total ammonia, nitrate, dissolved oxygen, water temperature, and pH);
- the raw water intake data for the Niagara Falls drinking water supply plant from 2016 to 2018 (*E. coli*); and
- TSS concentrations were obtained from the USGS for station 04216070 (Niagara River at Fort Erie) for the period 2014 to 2019.

Water quality data for the Niagara River at Fort Erie were obtained from the Environment Canada website while the water intake data was provided by Niagara Region.

The total phosphorus concentrations in the upper section of the Niagara River at Fort Erie (ON02HA0045) are compared to those on the lower section on Niagara-on-the-Lake (ON02HA0019) in Part I of the ACS concluding that current direct phosphorus loads to the Niagara River (e.g., not from Lake Erie) are not measurable. As a result, phosphorus was characterized using only the Niagara River at Fort Erie (ON02HA0045) dataset.

Measured data regarding TSS and $CBOD_5$ were not available in sufficient quantity to provide monthly characterization. However, since the water in the Niagara River is typically clear (NYPA 2005), it is expected that concentrations of TSS and $CBOD_5$ are low. Sixteen samples collected by the USGS provide annual estimates for the geometric mean and 75^{th} percentile TSS values of 5.2 mg/L and 11.3 mg/L, respectively.

A summary of the monthly water quality geo-mean and 75^{th} percentile values for Niagara River (ON02HA0045) and the raw water intake are presented in Table 4.

In general, water quality in the Niagara River meets all of the applicable objectives. Exceedances for the 75^{th} percentile were identified for total phosphorus for the period November to December, and *E. coli* for January and June to November. The highest monthly total phosphorus concentration typically occurs in December and January.

Measured data regarding TSS and $CBOD_5$ were not available in sufficient quantity to provide seasonal statistical summaries. However, since the water in the Niagara River is typically clear (NYPA, 2005), it is expected that concentrations of TSS and $CBOD_5$ are low. Sixteen samples collected by the USGS provide annual estimates for the geometric mean and 75^{th} percentile TSS values of 5.2 mg/L and 11.3 mg/L, respectively.

The GoldSim model uses the monthly 75^{th} percentile of ammonia, *E. coli*, nitrate, and total phosphorus.
For nitrate, the highest value from the Niagara River or the Raw Water Intake was applied yielding a conservative approach for prediction of assimilative capacity of the system. The assimilative capacity of the system for ammonia is based on the regulatory limit of unionized ammonia, ammonia in the system (based on 75^{th} percentile), and 75^{th} percentile values of pH and temperature.

Table 4: Summary of Monthly Water Quality Concentrations for Niagara River

Month	Station	Number of Samples [1]	Total Ammonia (mg/L)		Nitrate		*E. coli*		Phosphorus (mg/L)		Temperature (ºC)		pH	
			Geo-mean	75th	Geo-mean	75th	Geo-mean	75th	Geo-mean	75th	Geo-mean	75th	Geo-mean	75th
January	Niagara River	247-78	0.01	0.01	0.26	0.32	--[2]	--[2]	**0.030**	**0.046**	0.07	0.67	7.95	8.10
	Raw Water Intake	41	--[2]	--[2]	0.20	0.28	6	11	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]
February	Niagara River	226-69	0.01	0.01	0.26	0.31	--[2]	--[2]	0.021	**0.031**	0.06	0.25	8.06	8.18
	Raw Water Intake	36	--[2]	--[2]	0.40	0.54	6	10	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]
March	Niagara River	297-75	0.01	0.02	0.26	0.29	--[2]	--[2]	0.019	0.025	0.74	2.49	7.93	8.10
	Raw Water Intake	38	--[2]	--[2]	0.24	0.26	3	4	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]
April	Niagara River	298-47	0.03	0.06	0.26	0.30	--[2]	--[2]	0.020	0.026	4.40	7.82	8.06	8.10
	Raw Water Intake	38	--[2]	--[2]	0.15	0.19	4	6	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]
May	Niagara River	292-54	0.03	0.05	0.26	0.32	--[2]	--[2]	0.018	0.026	11.68	14.07	8.12	8.20
	Raw Water Intake	39	--[2]	--[2]	0.24	0.30	2	3	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]
June	Niagara River	276-53	0.03	0.05	0.28	0.32	--[2]	--[2]	0.016	0.023	18.52	20.29	8.18	8.30
	Raw Water Intake	37	--[2]	--[2]	0.17	0.23	3	4	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]
July	Niagara River	285-56	0.02	0.04	0.19	0.23	--[2]	--[2]	0.015	0.021	23.16	24.45	8.31	8.40
	Raw Water Intake	41	--[2]	--[2]	0.14	0.18	3	4	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]
August	Niagara River	309-56	0.02	0.04	0.13	0.17	--[2]	--[2]	0.015	0.022	23.59	24.41	8.27	8.40
	Raw Water Intake	39	--[2]	--[2]	0.12	0.13	4	5	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]
September	Niagara River	299-58	0.03	0.04	0.12	0.16	--[2]	--[2]	0.016	0.021	21.19	22.51	8.23	8.30
	Raw Water Intake	39	--[2]	--[2]	0.11	0.12	4	9	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]
October	Niagara River	309-58	0.01	0.04	0.14	0.18	--[2]	--[2]	0.017	0.025	15.07	17.49	8.22	8.30
	Raw Water Intake	40	--[2]	--[2]	0.11	0.11	6	10	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]
November	Niagara River	271-73	0.01	0.02	0.17	0.22	--[2]	7.000	0.023	**0.033**	7.82	10.08	8.06	8.20
	Raw Water Intake	37	--[2]	--[2]	0.11	0.12	6	7	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]
December	Niagara River	274-76	0.01	0.02	0.23	0.30	--[2]	--[2]	**0.032**	**0.049**	1.91	5.18	7.99	8.10
	Raw Water Intake	38	--[2]	--[2]	0.15	0.19	4	8	--[2]	--[2]	--[2]	--[2]	--[2]	--[2]

Notes:
1. Range of number of samples collected for the period of record for Niagara River at ON02HA0045 (1981 to 1991) and Niagara Falls Watertrax (2016 to 2018) per the month of interest
2. No data available
3. Highlighted values correspond with input to the GoldSim model.
4. Bold values indicate exceedances of applicable PWQO.

2.2.4 Lyons Creek

A summary of measured water quality in Lyons Creek is provided in Table 5, containing the monthly water quality geo-mean and 75th percentile values for monitoring station LY003. Data were provided by NPCA for station LY003 between 2003 and 2018.

The flows in Lyons Creek are a combination of supplemental flows from the Welland Canal (which is effectively water from Lake Erie) and natural drainage from the lower section of the Lyon Creek Watershed. Water quality in Lyons Creek consistently exceeds the PWQO guidelines for phosphorus as expected for a small watershed that drains agricultural areas, and occasionally exceeds *E. coli*. CBOD data was available only for the 2009 to 2014 period, while DO and TSS were not available in the dataset provided for this study.

The GoldSim model uses the monthly 75th percentiles of ammonia, *E. coli*, nitrate, and total phosphorus. The assimilative capacity of the system for ammonia is based on the regulatory limit of unionized ammonia, ammonia in the system (based on 75th percentile), and 75th percentile values of pH and temperature.

Table 5: Summary of Monthly Water Quality Concentrations for Lyons Creek

Month	Station	Number of Samples [1]	Total Ammonia (mg/L)		Nitrate		*E. coli*		Total Phosphorus (mg/L)		$CBOD_5$		Water Temperature (ºC)		pH	
			Geo-mean	75th	Geo-mean	75th	Geo-mean	75th	Geo-mean	75th	Geo-mean	75th	Geo-mean	75th	Geo-mean	75th
January	LC003	3	0.06	0.06[3]	0.90	0.90[3]	**630**	**630[3]**	**0.280**	**0.280[3]**	1.32[4]	2.00[4]	0.30[5]	0.30[3]	7.38	7.38[3]
February	LC003	1	0.09[2]	0.20[2]	0.84	0.84[3]	**410**	**410[3]**	**0.230**	**0.230[3]**	1.74[4]	2.00[4]	3.38[5]	3.38[3]	7.03	7.03[3]
March	LC003	5	0.12	0.34	0.22	0.65	41	**110**	**0.123**	**0.150**	1.74[4]	2.00[4]	3.38	14.3[5]	7.70	7.89
April	LC003	15	0.06	0.12	0.09	0.20	56	**200**	**0.140**	**0.185**	1.74	2.00	6.42	14.7[5]	7.79	7.95
May	LC003	16	0.04	0.11	0.06	0.20	37	56	**0.112**	**0.130**	1.52	2.00	9.67	18.70	7.90	8.16
June	LC003	16	0.04	0.09	0.06	0.20	51	94	**0.153**	**0.208**	1.26	2.00	7.88	25.70	7.88	8.04
July	LC003	16	0.05	0.10	0.06	0.09	21	40	**0.151**	**0.168**	0.76	1.50	23.97	26.40	7.86	8.03
August	LC003	13	0.03	0.08	0.09	0.20	27	40	**0.116**	**0.145**	0.84	1.75	27.00	27.0[3]	7.87	8.00
September	LC003	16	0.03	0.07	0.09	0.20	28	66	**0.086**	**0.115**	0.76	1.00	23.76	25.10	7.75	7.96
October	LC003	14	0.04	0.09	0.11	0.21	58	**153**	**0.113**	**0.193**	1.43	2.50	21.92	25.30	7.82	8.02
November	LC003	14	0.04	0.10	0.12	0.24	56	90	**0.117**	**0.200**	1.32	2.00	12.65	23.40[5]	7.78	7.90
December	LC003	1	0.05[2]	0.08[2]	0.55	0.55[3]	10	10[3]	**0.049**	**0.050[3]**	1.32[4]	2.00[4]	0.30	0.30[3]	7.91	7.91[3]

Notes:
1. Total number of samples collected for the period of record (2003 to 2018) and month of interest for all parameters except CBOD5 and water temperature
2. Value calculated as average of previous and next month
3. Insufficient samples to develop a distribution. Value corresponds to geo-mean.
4. Insufficient samples to develop a distribution. Value corresponds to maximum monthly value.
5. No data for the month. Value corresponds to closer month with available data
6. Highlighted values correspond with input to the GoldSim model.
7. Bold values indicate exceedances of applicable PWQO.

2.2.5 Existing Niagara Falls Wastewater Treatment Plant, Primary Bypass, and Secondary Bypass

Water quality data and laboratory analysis were provided for the existing Niagara Falls WWTP final effluent from 2015 to 2018 by the Niagara Region.

The assimilative capacity of the system was estimated by excluding all CSOs, and assuming that the water quality from the effluent at the existing Niagara Falls WWTP corresponds with the regulatory limits outlined in the Amended Environmental Compliance Approval (ECA) number 7962-7ZLKR6, issued on February 3, 2010. The regulated parameters which are outlined in the aforementioned ECA are total phosphorus and *E. coli*, with effluent limits specified as at 0.75 mg/L and 200 counts/100 ml, respectively.

The historic monthly final effluent quality is summarized in Table 6.

Table 6: Summary of Monthly Final Effluent Quality Concentrations for Existing Niagara Falls WWTP

Month	Number of Samples[1]	Total Ammonia (mg/L)		Nitrate		*E. coli*		Total Phosphorus (mg/L)		$CBOD_5$		Water Temperature (ºC)		pH	
		Geo-mean	75th	Geo-mean	75th	Geo-mean	75th	Geo-mean	75th	Geo-mean	75th	Geo-mean	75th	Geo-mean	75th
January	124	4.35	10.13	6.12	9.68	7.4	11.5	0.298	0.355	4.12	5.65	9.22	10.88	7.27	7.38
February	113	3.62	9.60	6.93	9.29	4.6	10.0	0.310	0.410	4.82	6.23	9.25	10.87	7.25	7.33
March	124	3.96	8.78	5.67	8.46	9.1	9.5	0.267	0.350	4.72	6.48	9.47	10.97	7.26	7.38
April	120	2.32	6.28	5.60	8.20	10.7	22.0	0.237	0.300	4.46	6.00	11.69	12.84	7.32	7.47
May	124	2.66	7.10	6.46	9.69	7.4	11.0	0.346	0.410	4.97	6.80	15.20	16.64	7.29	7.40
June	120	3.07	8.43	4.49	7.43	5.5	8.0	0.396	0.483	5.20	6.90	18.30	19.50	7.26	7.40
July	124	4.01	9.26	5.86	7.52	7.8	13.0	0.389	0.528	4.38	6.38	20.88	22.01	7.27	7.40
August	124	3.99	7.43	5.85	8.02	6.0	9.0	0.417	0.570	6.08	10.65	21.65	22.66	7.20	7.30
September	120	3.53	7.81	6.23	8.20	7.1	10.0	0.444	0.598	6.84	11.45	20.88	22.35	7.25	7.34
October	124	3.63	8.58	4.96	7.49	7.3	10.0	0.349	0.420	6.02	9.63	17.53	19.24	7.25	7.30
November	120	3.93	7.78	6.10	8.12	13.4	34.0	0.263	0.333	4.19	6.00	14.28	15.48	7.22	7.32
December	124	4.16	8.79	6.64	9.76	11.3	17.0	0.285	0.360	4.29	5.40	11.99	13.70	7.25	7.35

Notes:
1. Total number of samples collected for the period of record (2015 to 2018)

2.3 Data Conclusions and Generalizations

Based on the preceding characterisation of available flow and water quality data, the following conclusions are provided with respect to the detailed assessment discharge of the effluent into Chippewa Creek:

- Flows in the HEPC and Chippewa Creek are controlled by the operation of the ICD and should not be represented as a natural flow regime in the ACS.
- The background concentrations of two parameters, phosphorus, and *E. coli* are shown to exceed their respective water quality criteria within two or more watercourses discharging to the HEPC.
- While the Niagara River generally has lower concentrations of phosphorus when compared to the Welland River and Lyons Creek, it represents a far more significant loading source of this parameter due to the considerable difference in flows directed through the HEPC from all sources:
  - Niagara River approximates 95.1% of background HEPC flows;
  - Welland River (natural and supplemental flows) approximates 4.5% of background HEPC flows;
  - Lyons Creek contributes less than 0.3% of background HEPC flows; and
  - Existing Niagara Falls WWTP approximates 0.1% of background HEPC flows.
- Total phosphorus concentrations within the Niagara River tend to increase substantially outside the growing season. During the winter months, the 75th percentile phosphorus concentration in the Niagara River are almost twice that of other months.
- Notably, it has recently been estimated that 57% of all phosphorus loads to Lake Ontario come from the Niagara River from upstream sources in Lake Erie (ECCC & USEPA 2018).
- The Welland River East and Lyons Creek also have some local influence, particularly in spring when background phosphorus loading to the HEPC from these two watercourses alone can exceed 20%.
- Water quality in Welland River East, particularly total phosphorus, deteriorates as the natural flows increase. This correlation is likely attributed to the increased influence of poor land management practices during rainfall runoff compared to the beneficial dilution effects of consistent, supplemental inflows from the Welland Canal via the Port Robinson Pumping Station, ports in the old siphon, and the Welland WWTP bypass under low flow conditions.
- Relative to the Niagara River, bacteriological concentrations in the Welland River and Lyons Creek are so high that the Welland River and Lyons Creek are the dominant sources of *E. coli* throughout the winter and spring to the HEPC, despite order of magnitude differences in flow volume.
- As such, much of the water quality issues in the system are currently being influenced by background contributions from Lake Erie and smaller watersheds located upstream of the HEPC.

3.0 MODELLING APPROACH AND RESULTS

The modelling approach was designed with the following objectives:

- Estimate the remaining capacity of the receiving waters to accept the proposed WWTP effluent flows without exceeding applicable guidelines on a monthly basis;
- Estimate the recommended effluent limits for the preferred discharge location to Chippewa Creek and compare those limits to feasible limits based on the available treatment technology; and
- Estimate the existing and future concentrations in the receiving waters for effluent discharge to Chippewa Creek based on the recommended effluent limits.

The modelling approach was consistent with the Screening Level ACS completed to evaluate the original four discharge location options (Appendix A). The following points summarize the approach:

- Given the complex and regulated hydrodynamic conditions in the system, a stochastic model (GoldSim) was used to complete the ACS for total phosphorus, total ammonia, nitrate, and fecal coliforms (*E. coli*). Estimates for unionized ammonia were calculated based on modelled ammonia and measured 75th percentile values for temperature and pH.
- To provide an alternate estimate of the assimilative capacity, a mass balance model was developed to estimate the maximum allowable effluent concentrations for total ammonia, unionized ammonia, nitrate, fecal coliforms (*E. coli*), and total phosphorus for conditions where all the flows in the study area were assumed to be representative of low-flow conditions (e.g., 7Q20 or minimum regulated flow).
- The assimilative capacity was assessed at two compliance points; a local compliance point that is immediately downstream of the proposed discharge in Chippewa Creek and a system compliance point in the HEPC downstream of the existing Niagara Falls WWTP to consider cumulative effects in the study area.
- For parameters associated with oxygen in the water (dissolved oxygen and $CBOD_5$), the maximum allowable effluent concentrations were estimated using a simplified and conservative dissolved oxygen mass balance model that included $CBOD_5$ decay at the local compliance point. The assessment of dissolved oxygen also considered oxygen consumption due to the nitrification of ammonia. The system compliance point was not evaluated as reaeration is expected in the HEPC due to current speeds.
- A simple mass balance model was used to estimate the maximum allowable effluent concentrations for TSS based on the CCME recommended maximum increase of 5 mg/L over the background conditions.

In addition to the assimilative capacity modelling, this document also includes a mixing zone assessment that provides a conceptual outfall design, predictions of the performance of the outfall under various seasonal and flow conditions, and predicted plume concentration profiles immediately downstream of the proposed outfall.

A schematic of the study area showing the location of the local and system compliance points is provided in Figure 3.

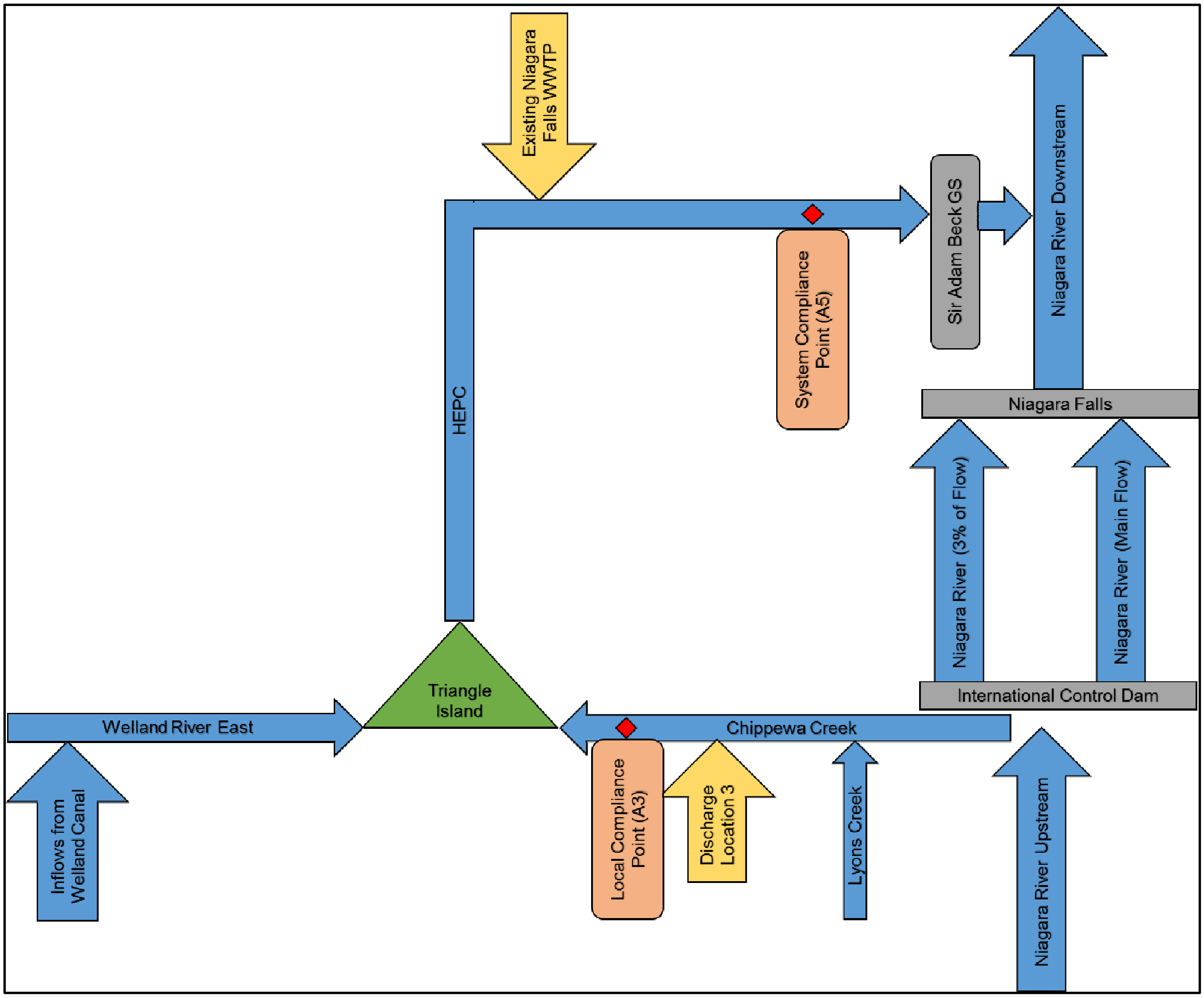


Figure 3: Local and System Compliance Points for Discharge at Location to Chippewa Creek

3.1 GoldSim Modelling

The stochastic water balance and water quality model developed for the Screening ACS using GoldSim was modified to use monthly input data (monthly flow distributions and monthly water quality data) instead of seasonal values. Technical details of the GoldSim software and model development are presented in detail in Appendix A. In GoldSim, conditional formatting was applied to the model compartments representing each month to become active only during the days corresponding to the specific month. The model was run stochastically using 1,000 iterations, for the modelling period which extended to a full year.

Flow and water quality data observed within the first and last day of each month were used to characterize flow and water quality for each specific month. Average, standard deviation, maximum and minimum flows were used to characterize monthly flow distributions for the Welland River East, Lyons Creek, and HEPC. Flows at the existing Niagara Falls WWTP were assumed as a constant value throughout the year. Water quality concentrations for inflows were based on the 75th percentile monthly concentrations from measured water quality data for total phosphorus, nitrate, total ammonia, and *E. coli*.

GoldSim was applied with the following objectives:

- Estimate the remaining capacity of the receiving waters to accept the proposed WWTP effluent flows without exceeding applicable guidelines on a monthly basis.
- To estimate the allowable effluent limits that will result in exceedances of the criteria no more than 5% of the time. The applicable water quality limits for phosphorus, nitrate, *E. coli*, and unionized ammonia were used by the model to calculate, for each constituent, the monthly mass allowed in the system based on input mass load from all sources and the regulatory limits. The model was run stochastically for 1,000 iterations which allowed the expression of the assimilative capacity results in terms of probability of exceedance. The capacity in the system was assessed for the local and system compliance points and included phosphorus, nitrate, *E. coli* and total ammonia. Allowable mass was then converted to the allowable concentration according to the flow in the new WWTP.
- To predict future phosphorus, nitrate, *E. coli*, and total ammonia concentrations at the local and system compliance points based on proposed effluent limits at the new WWTP. Future concentrations are expressed in probabilistic form on a monthly basis.

3.1.1 Flow Implementation

Flow was implemented in the model based on the available data and the stochastic modelling using the GoldSim model for Welland River East, Lyons Creek, and the HEPC. Flow in Chippewa Creek was estimated using the HEPC flow as well as the flows coming from the Welland River East and Lyons Creek (Sections 2.1.3 and 2.1.4).

3.1.1.1 Welland River East

Table 7 shows the parameters associated with the log-normal distributions developed to characterize the monthly flow in Welland River East in GoldSim. These distributions include all supplemental inflows from the Welland Canal into the Welland River East. Figure 4 and Figure 5 show the probability distribution of monthly flows.

Table 7: Summary of Monthly Flow Statistics for Welland River East Including Supplemental Flows

Parameter	Mean Flow (m^3/s)	Standard Deviation (m^3/s)	Maximum Flow (m^3/s)	Minimum Flow (m^3/s)
January	20.58	16.68	177.90	12.83
February	22.37	23.18	244.81	12.21
March	32.53	26.98	289.10	13.59
April	27.33	21.65	240.58	14.77
May	18.88	13.15	137.07	14.04
June	16.39	7.04	136.11	14.52
July	15.60	3.57	70.11	14.61
August	15.47	3.14	64.00	14.51
September	16.14	6.08	130.31	14.44
October	17.43	8.78	176.05	14.40
November	21.30	14.78	166.79	14.87
December	24.55	19.81	250.99	14.96

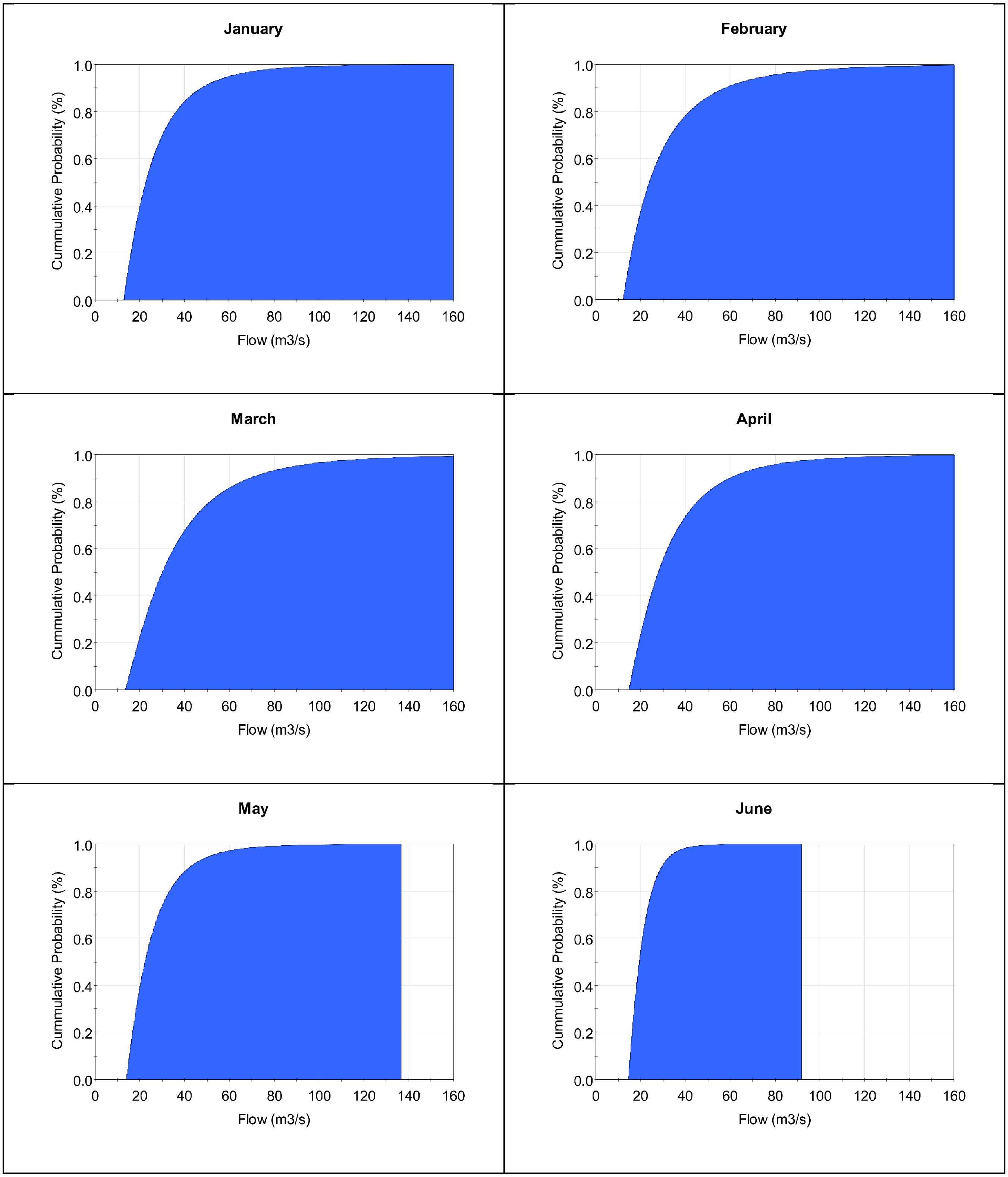


Figure 4: Monthly Log-Normal Distribution of Flows in the Welland River East Including Supplemental Inflows (January to June)

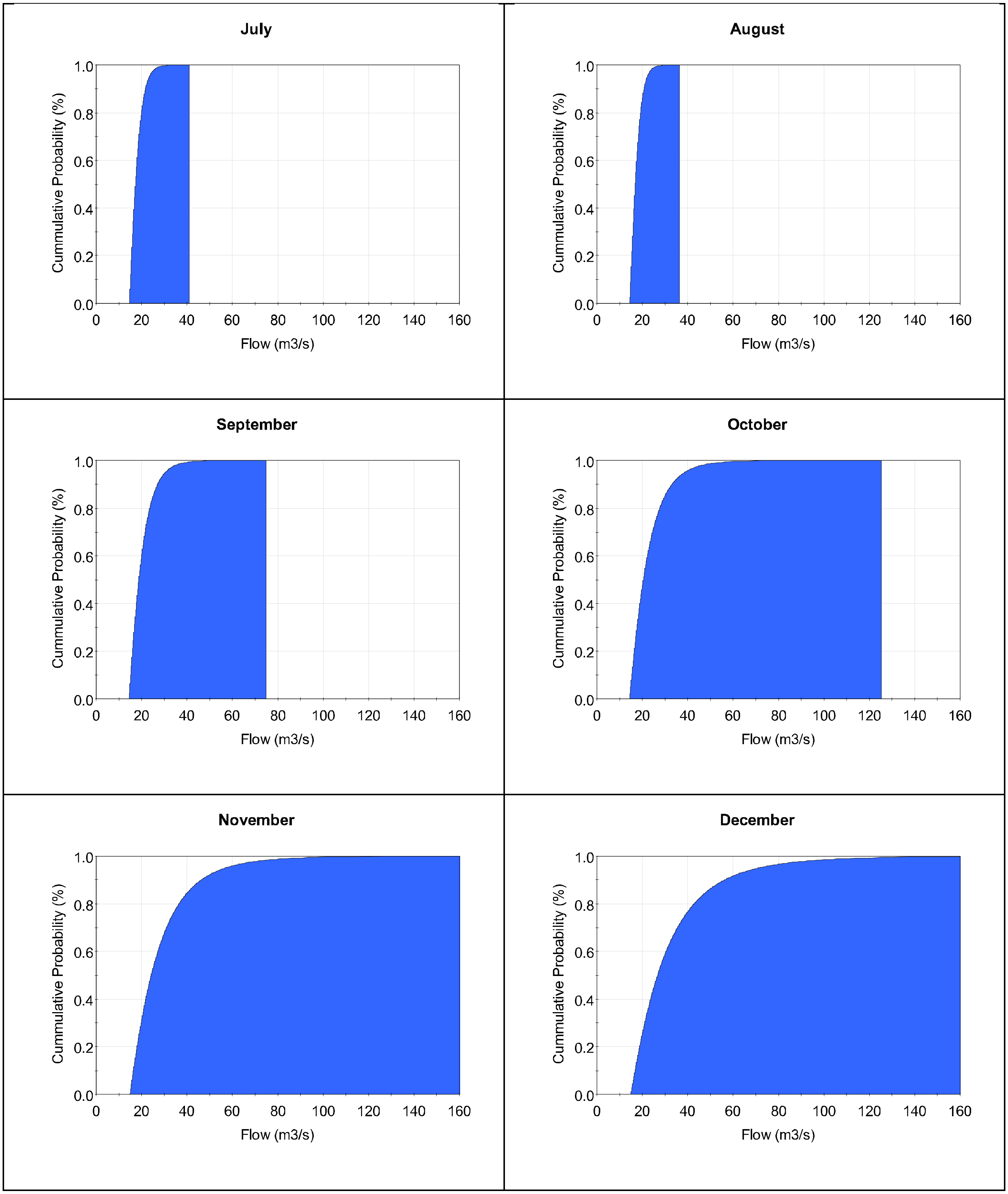


Figure 5: Monthly Log-Normal Distribution of Flows in the Welland River East Including Supplemental Inflows (July to December)

3.1.1.2 Lyons Creek

Table 8 shows the parameters associated with the monthly log-normal distributions developed to characterize the flow in Lyons Creek in GoldSim. Figure 6 and Figure 7 show the probability distribution of monthly flows.

Table 8: Summary of Monthly Flow Statistics for Lyons Creek

Parameter	Mean Flow (m^3/s)	Standard deviation (m^3/s)	Maximum Flow (m^3/s)	Minimum Flow (m^3/s)
January	1.21	2.30	22.95	0.14
February	1.55	3.20	32.27	0.14
March	2.76	3.73	38.20	0.14
April	2.05	2.99	31.51	0.31
May	0.95	1.82	17.28	0.28
June	0.54	0.97	17.08	0.28
July	0.42	0.49	7.95	0.28
August	0.42	0.43	7.12	0.28
September	0.52	0.84	16.29	0.28
October	0.70	1.21	22.61	0.28
November	1.17	2.04	21.27	0.28
December	1.47	2.74	32.75	0.14

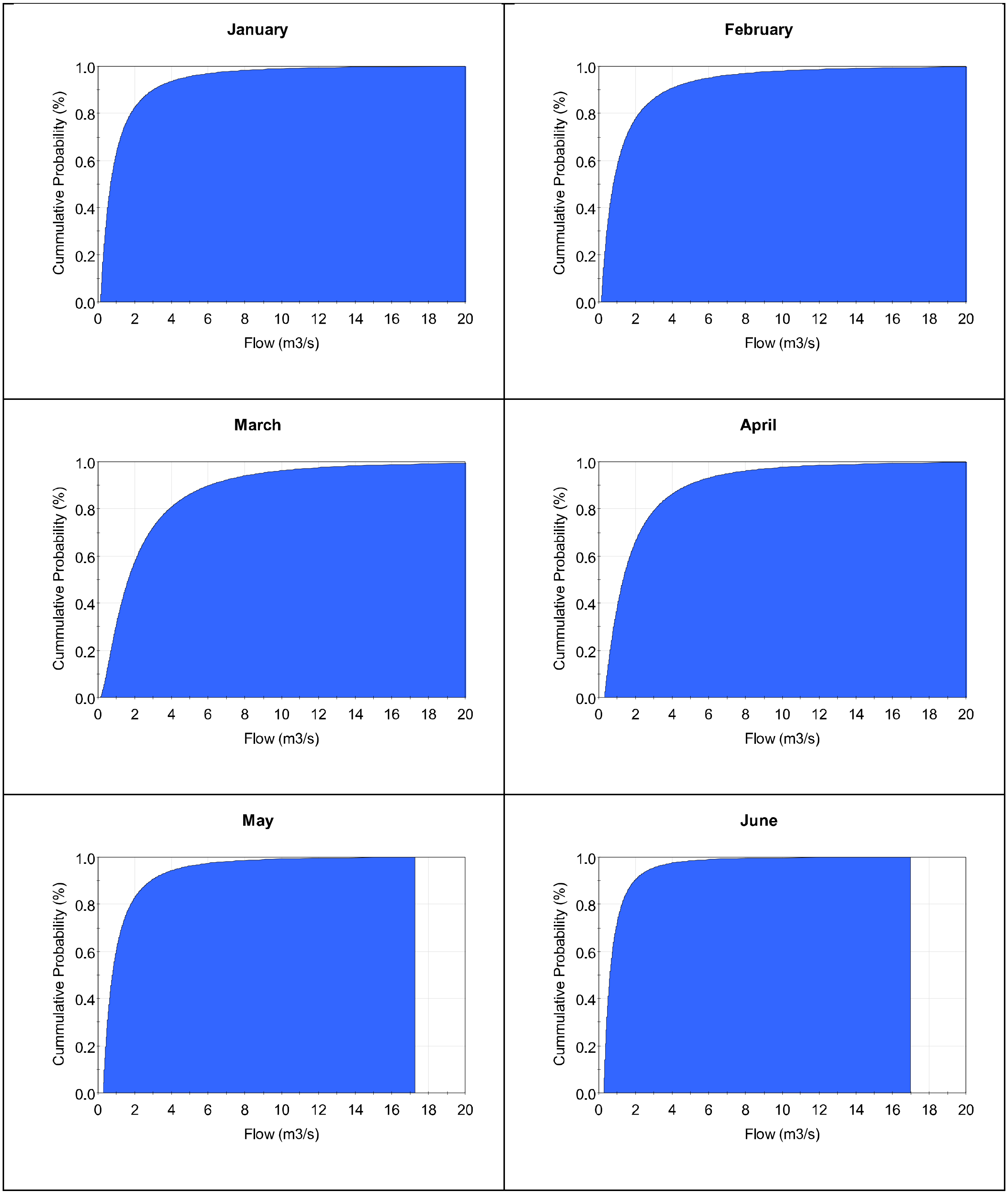


Figure 6: Monthly Log-Normal Distribution of Flows in Lyons Creek Including Supplemental Inflows (January to June)

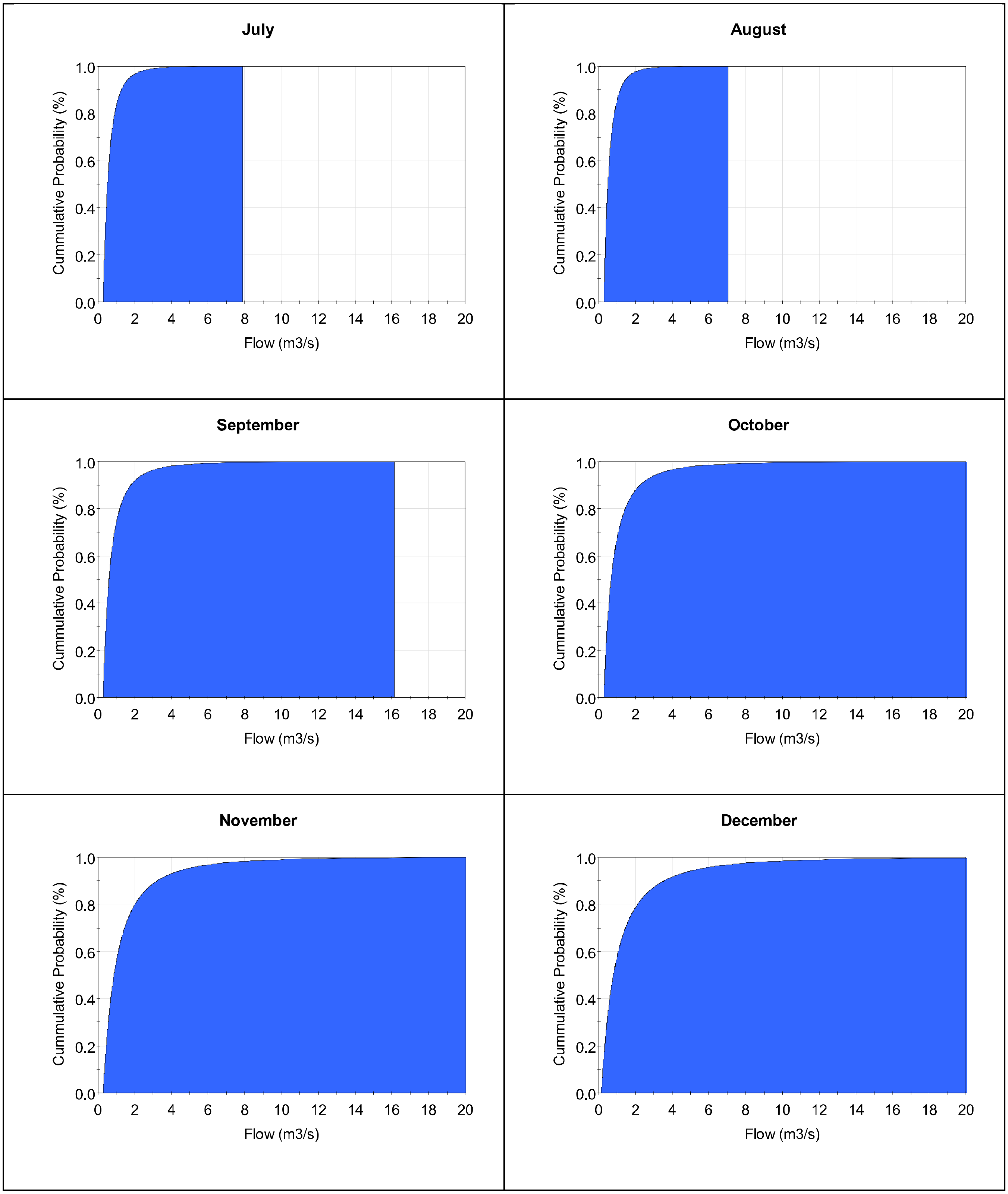


Figure 7: Monthly Log-Normal Distribution of Flows in Lyons Creek Including Supplemental Inflows (July to December)

3.1.1.3 Hydro Electric Power Canal (HEPC)

Table 9 shows the parameters associated with the log-normal distributions followed to characterize the monthly flow in HEPC in GoldSim. Figure 8 and Figure 9 show the probability distribution of monthly flow. In GoldSim, the flow through Chippewa Creek was calculated based on the difference between the flow in the HEPC and the corresponding flow in Welland River East and Lyons Creek.

Table 9: Summary of Monthly Flow Statistics for the Hydro Electric Power Canal

Parameter	Mean Flow (m^3/s)	Standard Deviation (m^3/s)	Maximum Flow (m^3/s)	Minimum Flow (m^3/s)
January	435	46.7	546	343
February	429	46.5	555	351
March	407	38.1	539	351
April	416	45.9	557	350
May	412	29.0	506	361
June	425	35.4	510	363
July	456	42.7	558	374
August	458	41.6	551	371
September	438	43.5	541	364
October	407	23.8	476	358
November	417	37.3	501	347
December	444	59.3	562	329

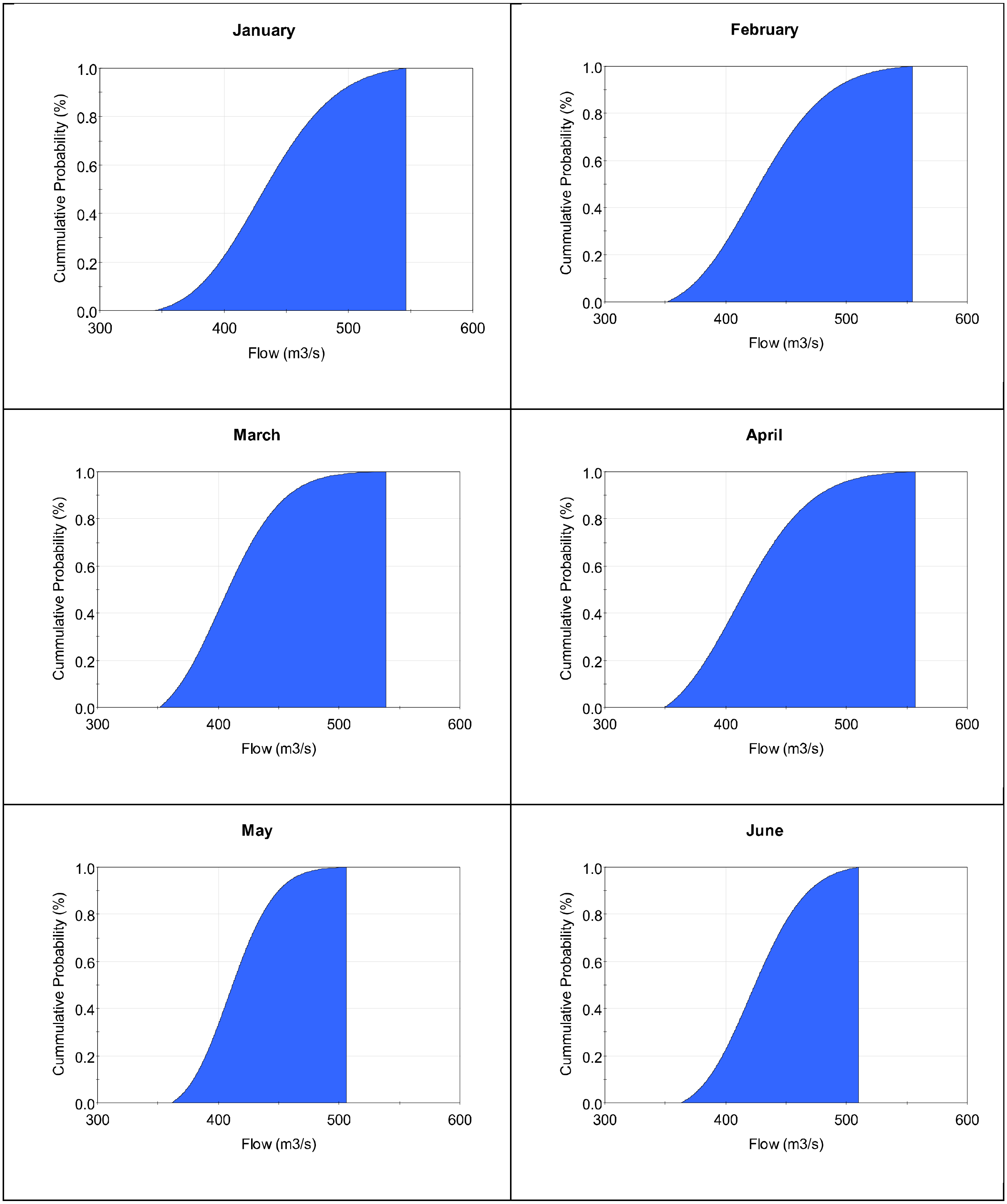


Figure 8: Monthly Log-Normal Distribution of Flows in HEPC (January to June)

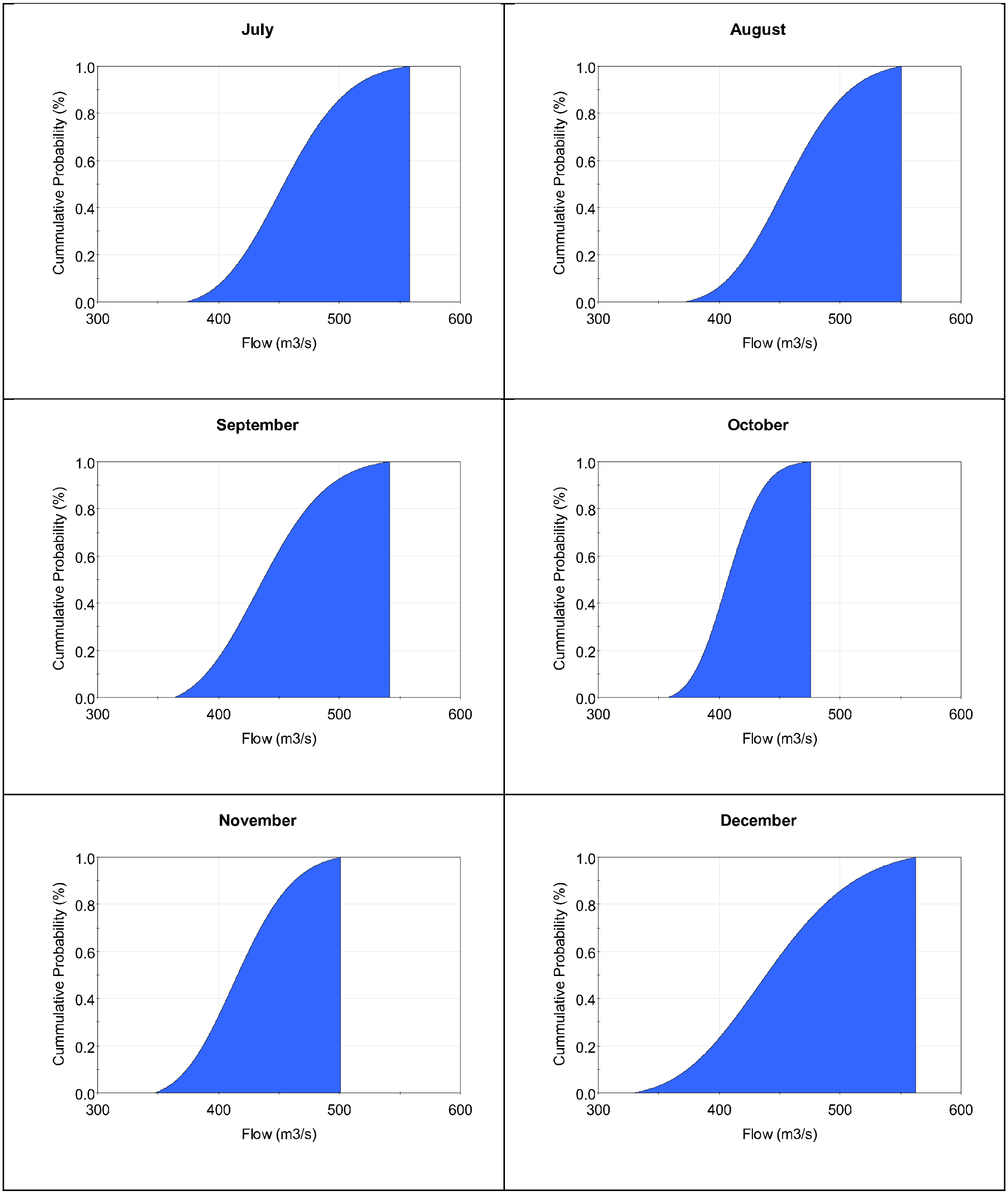


Figure 9: Monthly Log-Normal Distribution of Flows in HEPC (July to December)

3.1.1.4 Existing Niagara Falls Wastewater Treatment Plant

The effluent flow rate from the existing Niagara Falls WWTP was assumed to be equal to the rated capacity listed in the ECA of 68.3 MLD (0.79 m^3/s).

3.1.1.5 New South Niagara Wastewater Treatment Plant

Flow from proposed WWTP was assumed to be constant at 0.347 m^3/s (30,000 m^3/d).

3.1.2 Water Quality Implementation

The available data for water quality included ammonia, *E. coli*, nitrate, and total phosphorus. Water quality data associated with the 75th percentile was used for all inputs to the model, with the exception of the effluent from the existing Niagara Falls WWTP, which considered water quality as per the ECA regulatory limits for total phosphorus and *E. coli*.

3.1.3 Water Quality Objectives

The allowable effluent concentration for the proposed WWTP were estimated by calculating the mass allowed in the system until reaching applicable water qualitive objectives. The threshold for *E. coli*, total phosphorus, and nitrate were based on the guidelines provided in Table 2.

GoldSim does not incorporate accurate modelling of pH and water temperature. The fraction of the total ammonia that is unionized is a function of pH and temperature. The monthly target values for total ammonia were back calculated from the PWQO limit of 0.0164 mg/L as nitrogen for unionized ammonia based on the monthly 75th percentile water temperature and pH in Chippewa Creek and the HEPC.

The monthly thresholds for total ammonia, *E. coli*, nitrate, and total phosphorus in the receiver used to estimate recommended effluent limits are summarized in Table 10.

Table 10: Summary of Monthly Water Quality Objectives used in GoldSim

Month	Water Temperature (ºC)[1]	pH[1]	Total Ammonia (mg/L)[2]	*E. coli* (cfu/100ml)	Nitrate (mg/L)	Total Phosphorus (mg/L)
January	0.7	8.10	1.51	100	3	0.03
February	0.3	8.10	1.57	100	3	0.03
March	3.2	8.20	0.98	100	3	0.03
April	7.9	8.18	0.71	100	3	0.03
May	14.1	8.20	0.42	100	3	0.03
June	20.3	8.30	0.22	100	3	0.03
July	24.5	8.40	0.14	100	3	0.03
August	24.4	8.40	0.14	100	3	0.03
September	22.5	8.30	0.19	100	3	0.03
October	17.5	8.30	0.27	100	3	0.03
November	10.1	8.20	0.57	100	3	0.03
December	5.2	8.10	1.05	100	3	0.03

Note:
1. Measured 75th percentile value for either Niagara River or HEPC. Values used represented the conditions that resulted in the highest fraction of unionized ammonia.
2. Total ammonia criteria based on target unionized ammonia concentration of 0.0164 mg/L as N and seasonal average water temperature and pH in receiving water.

3.1.4 Maximum Allowable Effluent Concentrations

The allowable mass modelled in the system was extracted for the local compliance point (immediately downstream of the preferred discharge location) and at the system compliance point (downstream of the existing Niagara Falls WWTP). The recommended effluent concentrations were calculated by dividing the allowable mass by the flow from new WWTP. Large values in the table can be explained by the small flow rate in the proposed WWTP compared to the other flows in the system.

Table 11 shows the recommended effluent limits based on assimilative capacity at the local and system compliance points. These concentrations were calculated based on the GoldSim predictions for the 5% probability of exceedance.

These modelling results show that the system is currently at capacity for *E. coli* at the system compliance point from November to March and in September primarily due to contributions from the Welland River East. Elevated total phosphorus concentrations in the Niagara River from November and February result in no additional capacity for phosphorus at the local and system compliance points in those months. There are additional constraints on capacity to receive phosphorus at the system compliance point from March to June due to contributions from the Welland River East and the existing Niagara Falls WWTP.

Table 11: Summary of Maximum Allowable Effluent Concentrations from GoldSim Modelling

Month	Total Ammonia (mg/L)		*E. coli* (cfu/100ml)[1]		Nitrate (mg/L)		Total Phosphorus (mg/L)[1]	
	Local	System	Local	System	Local	System	Local	System
January	1,467	1,554	79,518	nc	2,530	2,594	nc	nc
February	855	919	75,315	nc	2,186	2,302	nc	nc
March	617	546	86,722	nc	2,457	2,687	2.7	nc
April	361	361	92,226	67,543	2,670	2,816	2.2	nc
May	174	154	98,596	97,296	2,725	2,851	3.2	nc
June	95	66	103,967	97,529	2,910	2,965	7.6	nc
July	102	80	104,734	102,999	3,025	3,139	9.9	6.5
August	151	135	98,330	95,695	2,933	3,077	8.2	5.4
September	225	213	90,825	nc	2,837	3,010	8.1	3.9
October	512	529	85,688	10,447	2,702	2,840	3.3	nc
November	940	1,016	85,288	nc	2,553	2,704	nc	nc
December	947	1,002	84,521	nc	2,484	2,602	nc	nc

Note:

1. "nc" denotes no capacity since existing background water quality exceeds (PWQO or CCME).

3.2 Mass Balance Modelling

A secondary verification to the GoldSim model results, mass balance modelling was completed using 75th percentile background water quality concentrations and minimum supplemental flows. Mass balance modelling estimated the maximum allowable effluent concentrations for total phosphorus, *E. coli*, nitrate, total ammonia, $CBOD_5$, and TSS, and the minimum dissolved oxygen concentration. The mass balance models generally followed the same structure as the GoldSim model as shown on Figure 3 and provided monthly estimates. One mass balance model was developed to assess total phosphorus, ammonia, nitrate, and *E. coli* such that both the local and system compliance points could be considered. Because dissolved oxygen and $CBOD_5$ are not independent, a specific mass balance model was developed for these two parameters simultaneously. A third mass balance model was developed for TSS since the water quality guideline for that parameter is based on an increase over ambient.

These models are intended to provide a secondary verification of the results provided by GoldSim by estimating the maximum allowable effluent concentrations for the worst-case conditions. The worst-case conditions were assumed to be the monthly cases where the low-flow conditions in each of the waterbodies occurred simultaneously.

The following points outline the inputs into the mass balance modelling:

- Total phosphorus, nitrate*, E. coli*, total ammonia, unionized ammonia, and TSS were modelled as conservative parameters and used the water quality limits provided in Table 2.
- The monthly maximum allowable effluent concentrations for total ammonia were estimated based on the maximum allowable unionized ammonia concentration and 75th percentile values for water temperature and pH.
- The discharge of effluent from the existing Niagara Falls WWTP was assumed to be the rated capacity (68.3 MLD).
- The effluent discharge rate from the proposed WWTP was 30 MLD.
- Inflow concentrations from the Niagara River, Lyons Creek, and Welland River East were assumed to be equal to the 75th percentile of the monthly concentrations.
- Where applicable, the existing effluent limits for the existing Niagara Falls WWTP were used (total phosphorus and *E. coli*).
- Since there are no effluent limits for the existing Niagara Falls WWTP for nitrate or ammonia, monthly 75th percentile values based on measured data were used (Section 2.2.5).
- The effluent from both the existing Niagara Falls WWTP and the proposed plant was assumed to mix completely in the receiving water immediately after release.
- Natural flows in the Welland River East were assumed to be negligible. The low-flow conditions in the Welland River East were assumed to be equal to the minimum supplemental flows from the Welland Canal as provided in Table 1.
- Inflows from Lyons Creek were assumed to be equal to the pumping rates from the Welland Canal since naturally occurring low-flow conditions (e.g., 7Q20) are negligible (Section 2.1.3).
- Flows in the HEPC were assumed to be equal to the 5th percentile of the monthly daily average flows in the HEPC based on data provided by OPG between 2016 and 2018.
- Flow into Chippewa Creek from the Niagara River was assumed be the same as the flow in the HEPC less the contributions from the Welland River East and Lyons Creek.

The assumed low-flow conditions used in the mass balance modelling are provided in Table 12.

Table 12: Summary of Low-Flow Conditions Used in Mass Balance Modelling

Month	HEPC[1] (m³/s)	Welland River			Lyons Creek			Chippewa Creek	
		Natural[2] (m³/s)	Pumped[3] (m³/s)	Total (m³/s)	Natural[2] (m³/s)	Pumped[4] (m³/s)	Total (m³/s)	Mouth[5] (m³/s)	Discharge[6] (m³/s)
January	370.6	0.022	12.2	12.2	0.003	0.140	0.143	358.3	358.4
February	364.5	0.018	11.4	11.4	0.003	0.140	0.143	353.0	353.1
March	360.4	0.015	12.7	12.7	0.002	0.140	0.142	347.6	347.7
April	376.8	0.332	13.9	14.3	0.046	0.280	0.326	362.2	362.5
May	375.8	0.041	13.0	13.0	0.006	0.280	0.286	362.5	362.8
June	375.1	0.000	14.2	14.2	0.000	0.280	0.280	360.6	360.9
July	389.7	0.000	14.5	14.5	0.000	0.280	0.280	375.0	375.3
August	384.6	0.000	14.2	14.2	0.000	0.280	0.280	370.1	370.4
September	377.8	0.000	14.1	14.1	0.000	0.280	0.280	363.4	363.7
October	369.3	0.000	14.0	14.0	0.000	0.280	0.280	355.0	355.3
November	358.0	0.000	14.5	14.5	0.000	0.280	0.280	343.2	343.5
December	356.2	0.038	14.8	14.9	0.005	0.140	0.145	341.2	341.3

Notes:

1. Estimate of low-flow condition in HEPC equal to 5th percentile of average daily flows
2. Estimated monthly 7Q20 flow from runoff.
3. Sum of all supplemental flows into Welland River East from Welland Canal (SLSMA 2019).
4. Estimated supplemental pumping rate from Welland Canal into Lyons Creek.
5. Estimated flow into Chippewa Creek from Niagara River (HEPC flow less flow from Welland River East and Lyons Creek).
6. Estimated flow in Chippewa Creek at preferred discharge location (HEPC flow less flow from Welland River East).

3.2.1 Mass Balance Modelling for Total Phosphorus, Ammonia, Nitrate, and *E. coli*

Monthly maximum allowable effluent concentrations were estimated at the local compliance point (Chippewa Creek east of Triangle Island) as well as at the system compliance point below the existing Niagara Falls WWTP. The resulting estimates of the maximum allowable effluent concentrations are provided in Table 13.
The modelling results were generally similar to those form the GoldSim modelling and suggest that:

- Poor water quality from the Welland River East may limit the available capacity for *E. coli* at the system compliance point in January, March, and December.
- Elevated total phosphorus concentrations in the Niagara River from November to February may limit capacity in Chippewa Creek.
- High phosphorus loads from the Welland River East may also limit the available capacity at the system compliance point during the spring (March through June).
- Contributions from the existing Niagara Falls WWTP may limit the available capacity at the system compliance point (A5) during October.

Table 13: Summary of Maximum Allowable Effluent Concentrations from Mass Balance Modelling of Low-Flow Conditions

Month	Total Ammonia (mg/L)[1]		*E. coli* (cfu/100ml)		Nitrate (mg/L)		Total Phosphorus (mg/L)	
	Local	System	Local	System	Local	System	Local	System
January	1,510	1,518	91,711	nc	2,772	2,777	nc	nc
February	1,542	1,564	91,467	72,188	2,744	2,751	nc	nc
March	931	913	95,937	822	2,716	2,762	4.9	nc
April	658	663	97,791	89,654	2,825	2,887	3.8	nc
May	381	370	101,149	100,264	2,799	2,863	3.8	nc
June	172	155	99,801	94,869	2,785	2,828	7.7	0.4
July	98	78	103,824	102,346	2,997	3,091	9.9	6.5
August	95	81	101,943	100,079	3,024	3,122	8.9	6.2
September	151	133	95,634	31,610	2,974	3,064	8.9	6.0
October	230	216	92,334	71,848	2,890	2,939	5.3	0.1
November	528	525	92,034	2,717	2,752	2,791	nc	nc
December	990	997	90,788	nc	2,661	2,687	nc	nc

Note:
1. "nc" denotes no capacity since existing water quality exceeds applicable criteria.

3.2.2 Mass Balance Modelling for Dissolved Oxygen and $CBOD_5$

Since dissolved oxygen, the nitrification of ammonia, and $CBOD_5$ of the effluent and background water all affect the downstream dissolved oxygen concentrations, these two parameters must be assessed together and could not be represented in GoldSim. The downstream dissolved oxygen at any downstream location is determined by the mixed (effluent and river) concentration of dissolved oxygen and the amount of oxygen consumed by the $CBOD_5$ in the time taken to reach that location. Other factors that affect the downstream dissolved oxygen include surface reaeration and algal growth/decay.

The nitrification of ammonia was considered in this assessment as it is expected to consume oxygen downstream of the outfall. However, the following points outline the rationale as to why the effects of nitrification on dissolved oxygen were considered negligible:

- The conversion of ammonia to nitrate consumes oxygen at a rate of 4.572 mg of oxygen per mg of ammonia (as N).
- The maximum increase in total ammonia concentration in Chippewa Creek as a result of the proposed discharge is predicted to be 0.003 mg/L based on the recommended effluent limits (Section 4.7) .
- If all the ammonia is instantly converted to nitrate, the total dissolved oxygen downstream of the outfall would decrease by approximately 0.014 mg/L.

The assessment of dissolved oxygen and $CBOD_5$ provides a conservative estimate of allowable effluent concentrations based on the following assumptions:

- Although measurements of dissolved oxygen in the Niagara River and HEPC are frequently at or above saturation due to turbulent flow conditions that provide a high degree of surface reaeration, surface reaeration is not included in this assessment.
- Given the typical clarity of the water in Niagara River and HEPC, the effects of algae are assumed to be negligible and are not included in the assessment.
- Given the short retention time in the system (e.g., less than a few hours), it is expected that only a fraction of the $CBOD_5$ will be consumed before leaving the study area. This assessment assumes that 50% of the $CBOD_5$ from upstream sources and the effluent will be consumed before leaving the system.
- $CBOD_5$ data was not available for the Niagara River. As such a background $CBOD_5$ concentration of 2 mg/L was assumed based on the highest seasonal 75th percentile $CBOD_5$ concentration found for the Welland River East (Table 3). These upstream conditions were applicable to the discharges into Chippewa Creek and the Niagara River.
- Upstream $CBOD_5$ concentrations in the Welland River East were based on the 75th percentile of the measured data (2 mg/L) since insufficient data was available to estimate monthly values.
- Upstream dissolved oxygen concentrations in the Niagara River were based on the monthly 25th percentile of the measured data.
- Water temperatures (required to estimate dissolved oxygen saturation concentrations) were based on the monthly 75th percentile temperature values for the Niagara River.
- Given the high degree of surface reaeration in the HEPC, dissolved oxygen, and $CBOD_5$ were not assessed at the system compliance point (below existing Niagara Falls WWTP).
- The assessment was based on the dissolved oxygen criteria for warm water fisheries (47% of saturation below 20ºC and 4 mg/L above 20ºC).

The allowable effluent $CBOD_5$ concentration was estimated by re-arranging the following equation:

$$Q_d D_d = Q_r D_r - f Q_r B_r + Q_e D_e - f Q_e B_e$$

Where: Q_d downstream flow (m^3/s) equal to sum of upstream and effluent flows,
Q_r upstream flow (m^3/s),
Q_e effluent flow (m^3/s),
D_d downstream dissolved oxygen concentration (mg/L) equal to guideline,
D_r upstream dissolved oxygen concentration (mg/L),
D_e effluent dissolved oxygen concentration (mg/L),
B_r upstream $CBOD_5$ concentration (mg/L),
B_e effluent $CBOD_5$ concentration (mg/L), and
f fraction of CBOD5 consumed in study area (assumed to be 0.5).

Estimates of the allowable monthly effluent $CBOD_5$ concentrations are provided in Table 14 for three levels of effluent dissolved oxygen saturation (10%, 50%, and 90%). Allowable concentrations for $CBOD_5$ are all greater than the minimum standard limit for secondary treated effluent of 15 mg/L.

The results indicate that allowable $CBOD_5$ concentrations are not sensitive to the dissolved oxygen levels in the effluent. Therefore, effluent dissolved oxygen concentration equal to 50% of the saturation concentration is recommended. The corresponding allowable monthly effluent $CBOD_5$ concentrations will be carried forward in this assessment.

Table 14: Estimated Allowable Monthly $CBOD_5$ Concentrations Based on Effluent Dissolved Oxygen

Month	Allowable Effluent $CBOD_5$ Concentration		
	Eff DO = 10% Sat[1]	Eff DO = 50% Sat[1]	Eff DO = 90% Sat[1]
January	12,241	12,253	12,264
February	12,852	12,863	12,874
March	13,824	13,835	13,846
April	14,349	14,359	14,368
May	12,946	12,954	12,962
June	9,297	9,304	9,311
July	7,091	7,098	7,104
August	**5,869**	**5,876**	**5,882**
September	5,876	5,883	5,890
October	7,022	7,030	7,037
November	7,951	7,960	7,969
December	8,959	8,969	8,979

Note:
1. Dissolved oxygen concentration in effluent expressed as percent of saturation.
2. **Bold** values indicate maximum allowable effluent concentrations carried forward in assessment.

3.2.3 Mass Balance Modelling for Total Suspended Solids

The assessment of TSS was based on the annual 75th percentile of the measured data in the Niagara River (11.3 mg/L) because there was insufficient data to establish monthly or seasonal values. The assessment was based on an allowable increase of TSS of 5 mg/L over the background conditions.

The allowable effluent TSS concentration was estimated by re-arranging the following equation:

$$(Q_r + Q_e)(C_r + \Delta C) = Q_r C_r + Q_e C_e$$

Where: Q_r upstream flow (m^3/s),
Q_e effluent flow (m^3/s),
C_r upstream TSS (mg/L),
C_e effluent TSS (mg/L), and
ΔC allowable TSS concentration increase (5 mg/L).

The estimated allowable monthly effluent concentrations for TSS are provided in Table 15 and indicate that the allowable effluent TSS concentration show little variation through the year.

Table 15: Estimated Allowable Monthly Effluent Total Suspended Solids Concentrations

Month	Allowable Total Suspended Solids (mg/L)
January	5,178
February	5,102
March	5,023
April	5,241
May	5,241
June	5,213
July	5,420
August	5,350
September	5,254
October	5,133
November	4,963
December	**4,932**

Note:
1. **Bold** values indicate maximum allowable effluent concentrations carried forward in assessment.

3.3 Mixing Zone Assessment (CORMIX Modelling)

This section provides the modelling and analysis included in the mixing zone assessment for the preferred outfall location into Chippewa Creek and includes the following:

- Estimates of the required effluent dilution required to meet PWQOs in the effluent plume based on the recommended effluent limits and background water quality.
- Development of a conceptual design for the outfall that will provide adequate performance under a range of environmental conditions and effluent flow rates.
- Prediction of the performance of the outfall design in terms of downstream mixing and dilution of the effluent plume under design flow conditions.
- Completion of a sensitivity analysis of outfall performance for variations in effluent flow rate and creek flows.

The mixing zone assessment assumes that the effluent will be discharging at a design flow of 0.35 m^3/s (30 MLD). The effluent discharge rate is expected to range from 0.23 m^3/s (20 MLD) during low flow periods to 1.39 m^3/s (120 MLD) during rainfall events.

3.3.1 Modelling Approach for Mixing Zone

The Cornell Mixing Zone Expert System (CORMIX) model, recognized by US EPA for mixing zone analysis, was used to conduct the assessment of effluent discharge and mixing processes and to quantify the dilution and mixing characteristics in the immediate vicinity of the discharge.

3.3.2 Required Effluent Dilution

The required dilution to either meet the applicable criteria (PWQO or CCME) was estimated on a monthly basis using background water quality in Chippewa Creek (Section 2.2.3) and recommended effluent limits (Section 4.7). Because there is no criterion for $CBOD_5$, the corresponding required dilution to meet criteria could not be estimated. The results of this analysis are summarized in Table 16.

Table 16: Summary of Estimated Effluent Dilution to Meet Water Quality Criteria

Parameter	Criteria or PWQO (mg/L)	Required Dilution to Meet Criteria	
		Minimum	Maximum
Total Ammonia[1]	0.14 to 1.51	2.26:1	8.18:1
Nitrate[2]	3.00	6.99:1	7.35:1
E. coli[1]	100	2.03:1	2.12:1
Total Phosphorus[1]	0.03	na[5]	**196:1**
$CBOD_5$	na[6]	na[6]	na[6]
TSS[1,4]	12.43	3.27:1	

Notes

1. Criteria based on PWQO
2. Criteria based on CCME Guidelines
3. PWQO for total ammonia is based in monthly water temperature and pH using unionized criteria of 0.0164 mg/L as N for unionized ammonia.
4. PWQO for TSS based on 10% increase over background concentration
5. Not available for several months when background concentrations of total phosphorus exceed PWQO (0.03 mg/L)
6. Not available – no criteria for $CBOD_5$

With the exception of total phosphorus, all the parameters with an applicable criterion require an effluent dilution of less than 10:1 to meet the criterion. The required dilution for total phosphorus can be as high as 196:1. Based on the required dilution for total phosphorus, a required dilution of 200:1 was used in subsequent assessments to compare the outfall performance for various conditions. Additionally, a dilution of 20:1 was also used for comparison as it represents 10% of the maximum required dilution.

3.3.3 Conceptual Outfall Design

The preferred discharge location is from the south bank of Chippewa Creek. Based on surveyed transects (Golder 2019), the creek channel in the area of the outfall is effectively a constructed channel with a uniform width, depth and side slopes that follows the original path of the Welland River prior to the construction of the HEPC. The channel is approximately 100 m wide at the surface and has a maximum depth of 12.6 m. The side slopes are approximately 2:1 (horizontal:vertical). The depth averaged width of the channel is approximately 76 m and the cross-sectional are was estimated to be 959 m^2.

The following points provide details of the conceptual outfall design that is also shown on Figure 10:

- Multiport diffuser with three duckbill valve ports angled 45° above horizontal (θ).
- The diffuser length (L_D) is 24 m with 12 m spacings between the ports.
- The distance from riverbank for the first port is 20 m and the distance to the centre of the diffuser is 32 m (DISTB).
- The ports are located 0.5 m above the creek bed (h_0).
- The ports are oriented in a downstream direction (e.g., pointed in same direction as flow during normal operation of the ICD).
- The diffuser is oriented perpendicular to the shoreline and current direction.

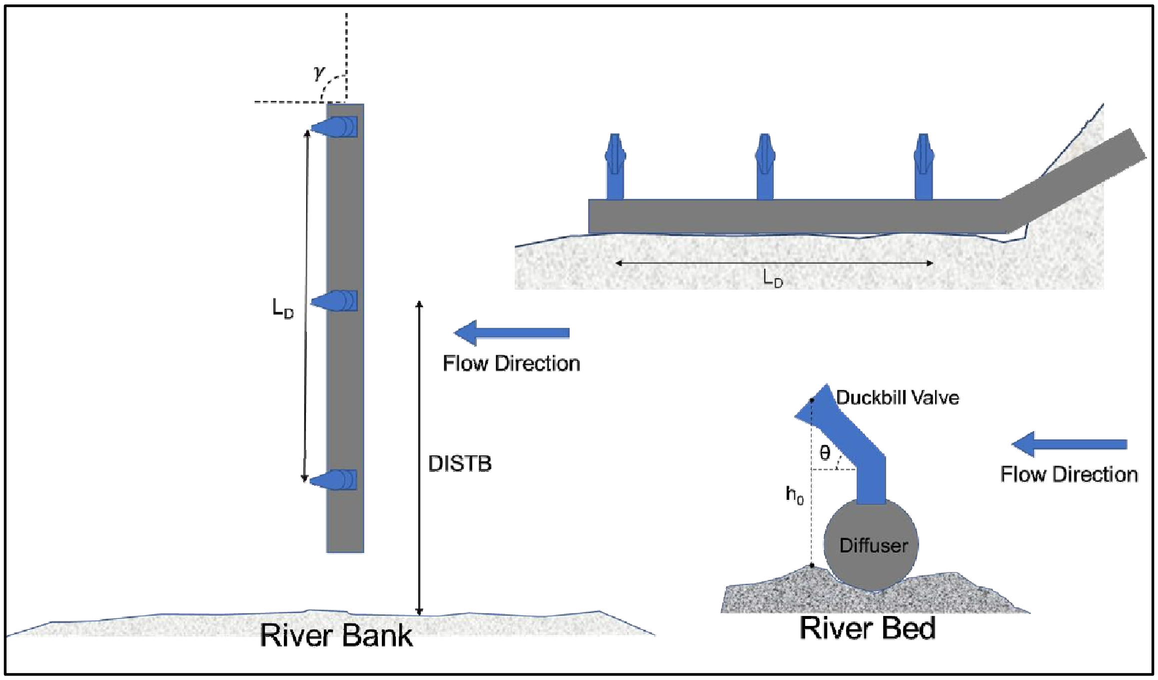


Figure 10: Schematic Views of The Multiport Diffuser

A TideFlex 250 duckbill valve outfall was selected for the conceptual design and is shown on Figure 11. Duckbill valves are made of flexible material that will generate variable effective cross section as a function of pressure and flow inside the duckbill valves, which provide higher jet exit velocities in low design flows and lower jet exit velocities in high design flows when compared to a conventional port. Duckbill valves also provide lower head losses than typical round ports that may be beneficial to the design of the treatment plant itself.

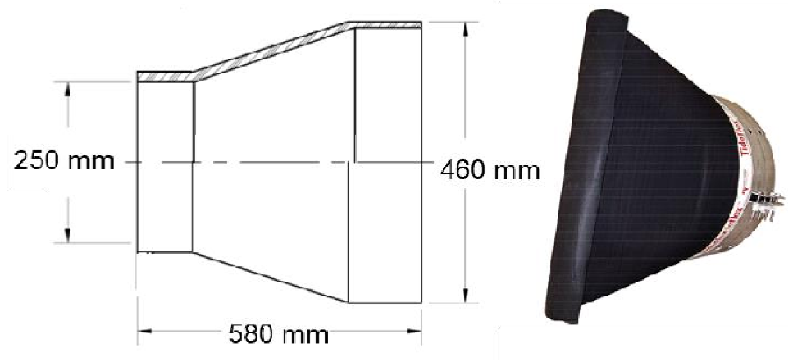


Figure 11: TideFlex 250 Duckbill Valve Dimensions

Wide bill TideFlex diffuser 250 characteristics such as jet exit velocity and total headloss are provided by TideFlex Technologies and shown on Figure 12.

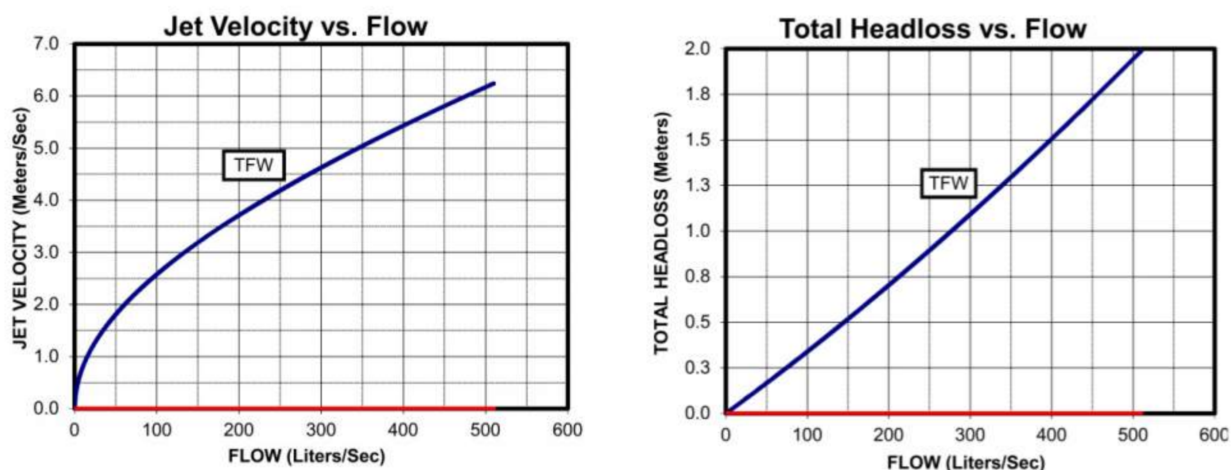


Figure 12: (a) Jet velocity and (b) Headloss for TideFlex 250 Duckbill Valve

While the design flow for the effluent is 30 MLD, the effluent flow rate is expected to vary from a low flow of 20 MLD up to 120 MLD for peak hourly flows. The operational parameters of the duckbill valve for the expected range of effluent flow rates are presented in Table 17.

Table 17: Jet Initial Flow Characteristics for TideFlex 250

Total flow		For Each TideFlex Port			
MLD	(m^3/s)	Flow (m^3/s)	Jet Velocity (m/s)	Total Headloss at Diffuser (m)	Effective Area (cm^2)
20	0.23	0.077	2.2	0.3	340
30	0.35	0.117	2.8	0.4	417
120	1.39	0.463	5.9	1.8	783

3.3.4 Selected Scenarios

This section outlines the selection of the scenarios used in the mixing zone assessment and considered the following factors:

- Expected flows from the proposed WWTP,
- Effluent buoyancy related effects based on water temperature and dissolved solids, and
- Expected range of flows in Chippewa Creek.

As stated earlier, the mixing zone assessment assumes that the effluent will be discharging at a design flow of 0.35 m³/s (30 MLD) but the effluent discharge rate is expected to range from 0.23 m³/s (20 MLD) during low flow periods to 1.39 m³/s (120 MLD) during rainfall events.

Monthly water temperatures for Chippewa Creek were estimated from data collected in the Niagara River (NOAA 9063020, 2007 to 2019) while the effluent temperatures for the new WWTP were based on recorded water temperatures from the existing Niagara Falls WWTP (2015 to 2018). The water temperatures used represent the average monthly value of the measured data. Monthly ambient water temperature varies from 0.4°C to 23.7C° from February to August and monthly effluent temperature changes from 9.5ºC to 21.7ºC from January to August as shown on Figure 13.

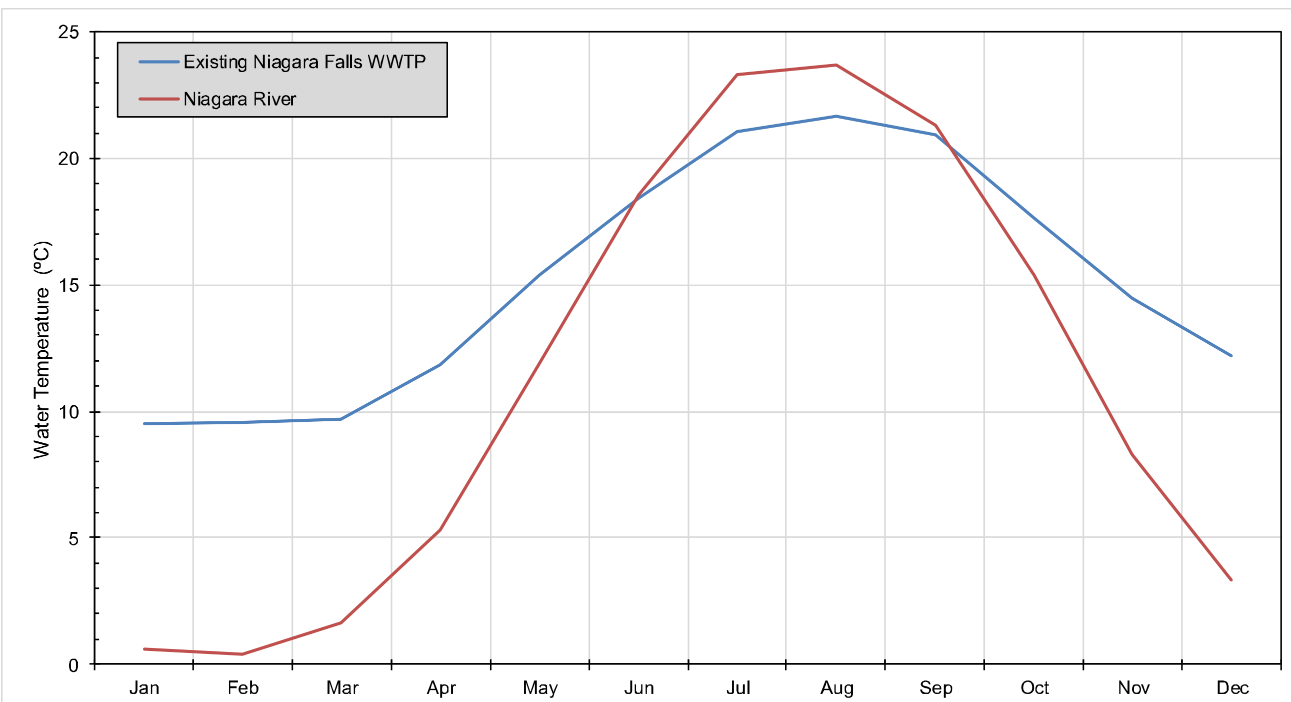


Figure 13: Monthly Temperatures for Effluent and Ambient Water

The dissolved solids concentrations in Chippewa Creek were based on conductivity measurements in the Niagara River (NOAA 9063020, 2007 to 2019). In general, the monthly average conductivities are consistent year-round and ranged from 277 to 295 µmhos/cm which correspond to dissolved solids concentrations that range from 154 to 164 mg/L. Neither dissolved solids nor conductivity data was available for the existing Niagara Falls WWTP. However, because the drinking water source for Niagara Falls is also the Niagara River, it was assumed that the dissolved solids in the effluent were the same as those in the Niagara River.

The densities of the effluent and Chippewa Creek were estimated based on the water temperatures and dissolved solids concentrations. Figure 14 shows the estimated monthly values for the ambient density (ρ_a) and effluent density (ρ_0). Density differences between the creek and effluent ($\rho_a - \rho_0$) show that from June to September the effluent is denser than the ambient water (negative value on Figure 15) which would result in an effluent plume that may have a tendency to sink to the bottom. However, the design of the outfall (e.g., upward orientation of ports and exit velocities) may be able to counteract some of the sinking tendencies.

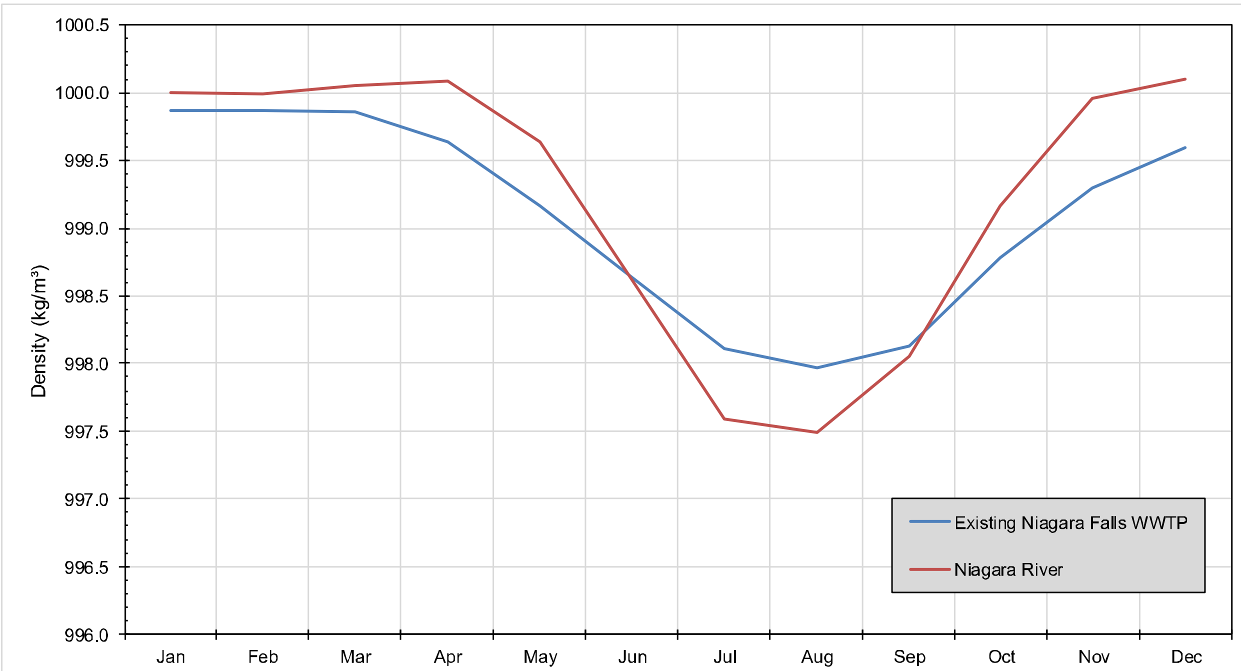


Figure 14: Monthly Density for Effluent and Ambient Water

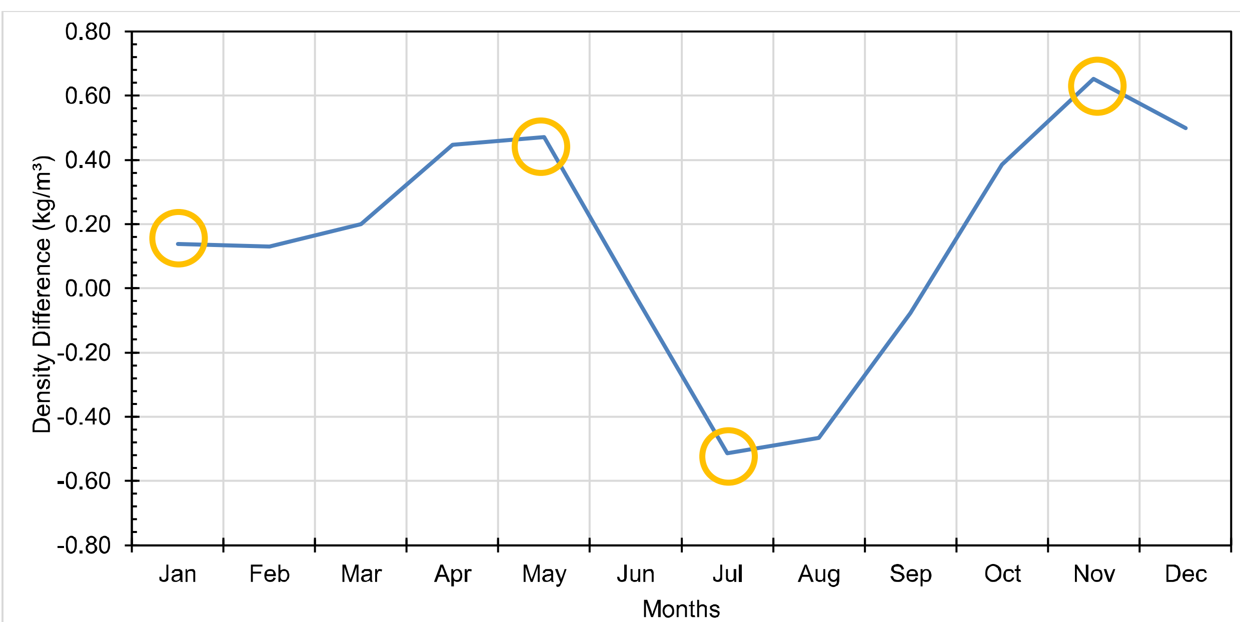


Figure 15: Ambient Water Density Difference with Respect to Effluent Density ($\rho_a - \rho_0$)

Flows in Chippewa Creek for the mixing zone assessment were estimated based on daily flows in the HEPC (2016 to 2018) less estimated inflows from the Welland River East and are summarized in Table 18.

Table 18: Estimated Monthly Flow Statistics for Chippewa Creek at Preferred Discharge Location

Parameter	Minimum Flow (m^3/s)	Average Flow (m^3/s)	Maximum Flow (m^3/s)
January	404	416	443
February	412	407	421
March	375	376	408
April	378	391	449
May	394	395	409
June	378	409	429
July	413	441	472
August	426	443	469
September	413	422	439
October	378	390	402
November	369	397	426
December	407	423	468

The primary scenarios for the assessment of the outfall design were selected to include a range of conditions that can be expected. Maximum absolute density difference in each season was selected (orange circles on Figure 15) to ensure that the selected scenarios included the critical conditions with respect to effluent buoyancy. January, May, and November were selected to represent the month where the effluent was most buoyant (e.g., effluent tends to float) while July was selected as the least buoyant month (e.g., effluent tends to sink). In all the primary scenarios, the minimum monthly flow and design effluent flows (30 MLD) were used. The monthly ambient and discharged effluent characteristics are summarized in Table 19.

Table 19: Summary of Ambient and Effluent Characteristics Used in Mixing Zone Assessment

Flow Information		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Ambient Conditions	Low Flow (m^3/s)	404	412	375	378	394	378	413	426	413	378	369	407
	Current Speed (m/s)	0.42	0.43	0.39	0.39	0.41	0.39	0.43	0.44	0.43	0.39	0.38	0.42
	Water Temperature (ºC)	0.6	0.4	1.6	5.3	11.9	18.6	23.3	23.7	21.3	15.4	8.3	3.4
	Conductivity (µS/cm@25C)	294.9	294.2	289.5	283.2	276.9	276.5	278.1	283.5	282.0	282.6	280.2	283.4
	Total Dissolved Solids (mg/L)	164.5	164.1	161.5	158.0	154.4	154.2	155.1	158.1	157.3	157.6	156.3	158.1
	Density (ρ_a) (kg/m^3)	1,000.01	999.99	1,000.05	1,000.08	999.63	998.61	997.59	997.50	998.05	999.17	999.96	1,000.09
Effluent Conditions	Water Temperature (ºC)	9.5	9.6	9.7	11.9	15.4	18.4	21.1	21.7	21.0	17.7	14.5	12.2
	Total Dissolved Solids (mg/L)	164.5	164.1	161.5	158.0	154.4	154.2	155.1	158.1	157.3	157.6	156.3	158.1
	Density (ρ_0) (kg/m^3)	999.87	999.87	999.85	999.64	999.16	998.64	998.10	997.96	998.13	998.78	999.30	999.60
Effluent Buoyancy	Density Difference ($\rho_a - \rho_0$) (kg/m^3)	0.14	0.13	0.20	0.45	0.47	-0.02	-0.51	-0.47	-0.08	0.39	0.65	0.50
	Buoyancy	Float	Float	Float	Float	Float	Sink	Sink	Sink	Sink	Float	Float	Float

3.3.5 Expected Outfall Performance

The performance and predicted downstream mixing of the plume for the primary scenarios was completed using CORMIX for a maximum downstream distance of 1,000 m. The results of the modelling are summarized in Table 20 and are presented graphically on Figures 16 to 18, respectively. The spatial extents of the January and July plumes are shown on spatial extent maps but were not prepared for May and November since they were similar to January. The following points summarize key results of the modelling:

- Scenarios with a floating plume (January, May, and November) had consistent results with a 200:1 dilution being reached in less than 5 m from the outfall.
- For the July scenario, CORMIX predictions in the turbulent mixing zone are for plumes from individual ports. The individual plumes become joined at the end of the turbulent mixing zone approximately 130 m downstream of the diffuser.
- For January, May, and November scenarios, the turbulent mixing zone is predicted to be 12 m in length and provide a dilution of greater than 340:1.
- For July, the turbulent mixing zone is predicted to be 133 m in length and provide a dilution of approximately 92:1. In July, a dilution of 200:1 is predicted to occur at a distance of just over 300 m.
- Beyond the turbulent mixing zone, mixing of the effluent is slower and is determined by the ambient conditions (passive mixing) in all the scenarios.
- For January, May, and November scenarios, the plume is expected to become vertically mixed with the ambient water at distances between 111 and 150 m.
- In July, the negative buoyancy of the plume (e.g., tendency to sink) is expected to cause the plume to remain vertically stratified in the bottom 2 m of the channel and travel along the channel bottom beyond a distance of 1,000 m.
- CORMIX does not predict the plume to become laterally well mixed within the modelled area.

Table 20: Summary of Mixing Zone Modelling for a Conceptual Outfall Design

Scenario	Turbulent Mixing Zone[1]			20:1 Dilution Plume		200:1 Dilution Plume		Distance to Vertically Mixed (m)
	Length (m)	Width (m)	Dilution	Length (m)	Width (m)	Length (m)	Width (m)	
Design Flow in January	12.0	24	350:1	0.1	24	4.0	24	111
Design Flow in May	12.0	24	342:1	0.1	24	4.1	24	130
Design Flow in July	133	4.2[2]	91.7:1	12.8	1.9[2]	328	47	-
Design Flow in November	12.0	24	342:1	0.1	24	4.1	24	150

Notes:

1. Turbulent mixing zone assumed to be the first output module from CORMIX and represents the mixing that is mostly influenced by the design of the outfall.
2. For the July scenario, CORMIX predictions in the turbulent mixing zone are for plumes from individual ports. The individual plumes become joined at the end of the turbulent mixing zone.

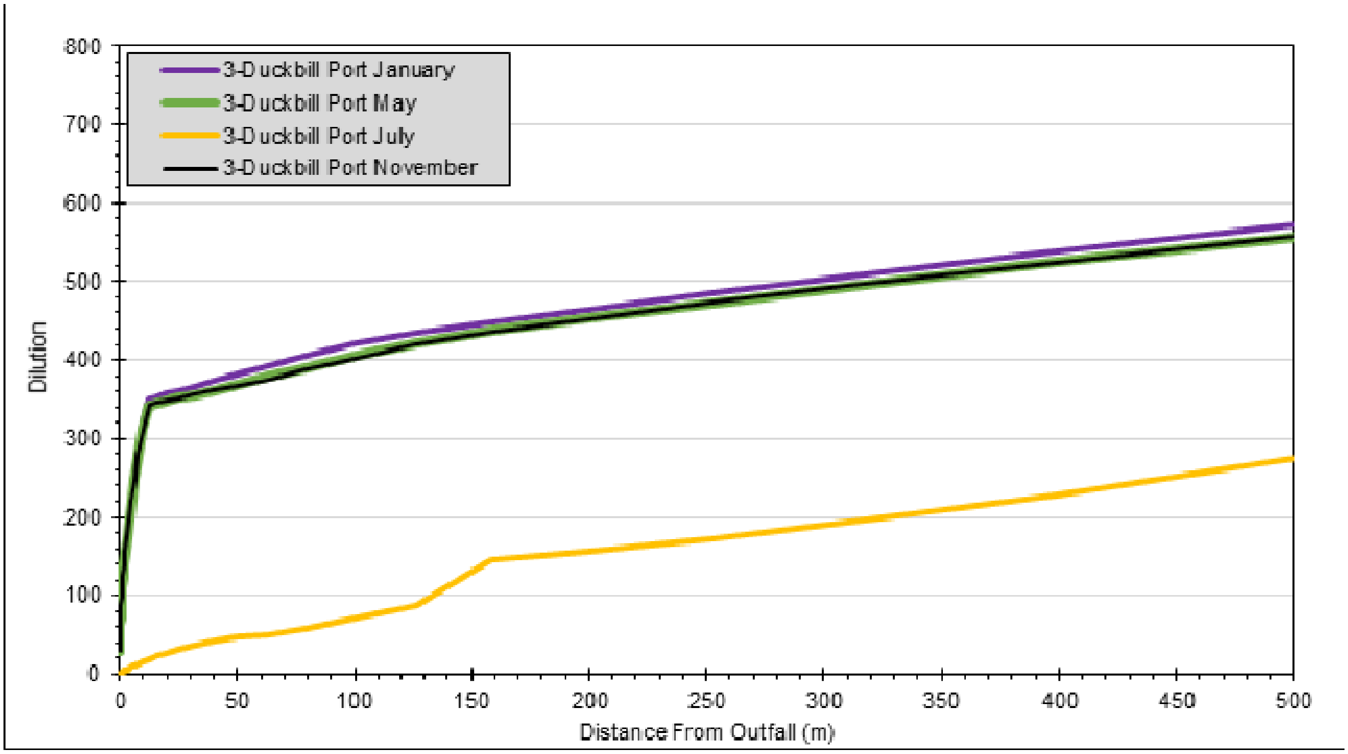
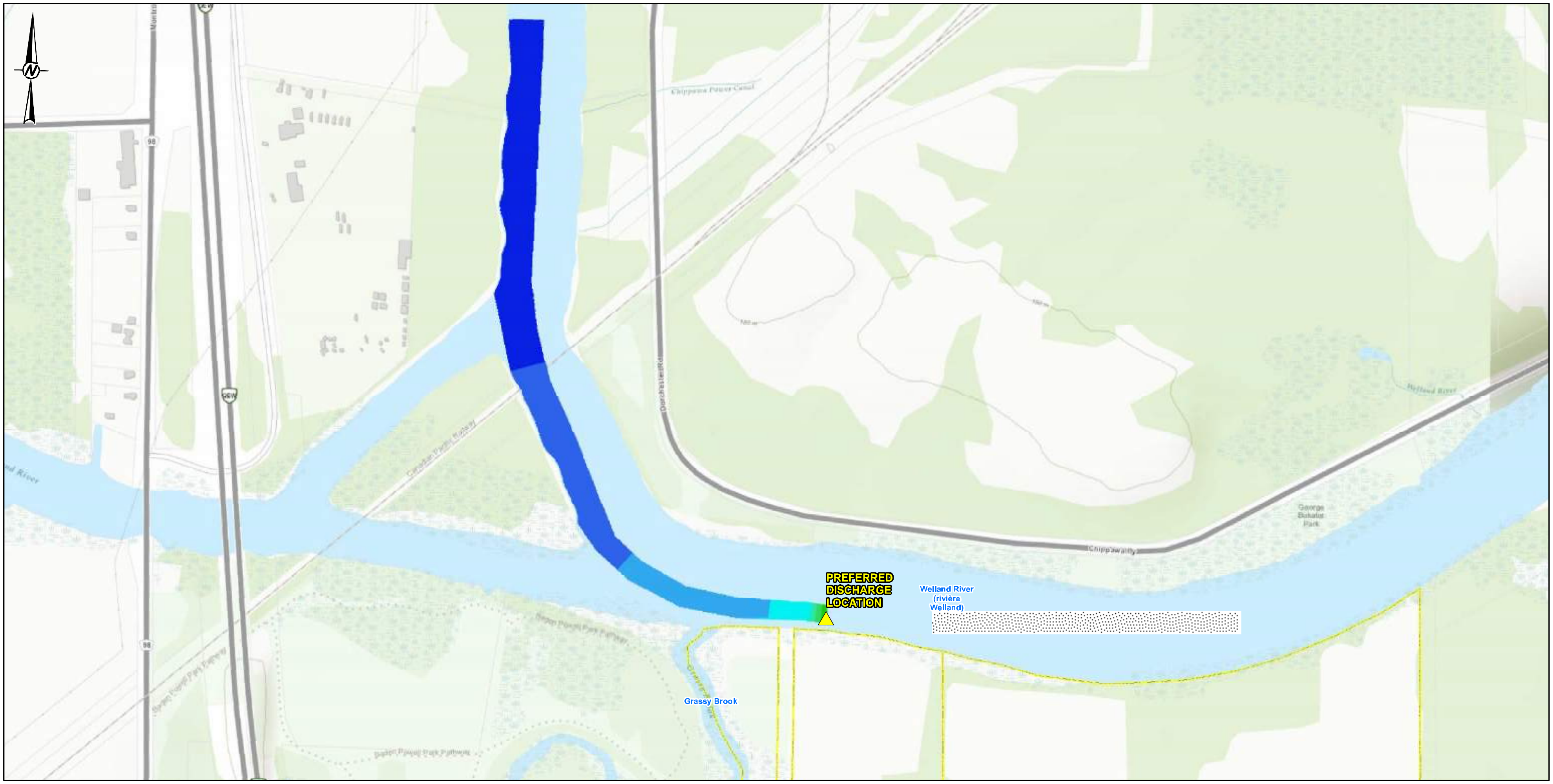


Figure 16: Distance-Dilution Plots for Primary Scenarios at 30 MLD



LEGEND

- PREFERRED DISCHARGE LOCATION
- WATERBODY
- PREFERRED SITE LOCATION 8

PLUME DILUTION

- 0 - 100
- 100 - 200
- 200 - 300
- 300 - 400
- 400 - 500
- 500 - 600
- 600- 700

0 100 200
1:5,000 METRES

REFERENCE(S)

1. BASE DATA - MNRF LIO, OBTAINED 2019
2. PRODUCED BY GOLDER ASSOCIATES LTD UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES AND FORESTRY, © QUEENS PRINTER 2019
3. SOURCES: ESRI, HERE, GARMIN, INTERMAP, INCREMENT P CORP., GEBCO, USGS, FAO, NPS, NRCAN, GEOBASE, IGN, KADASTER NL, ORDNANCE SURVEY, ESRI JAPAN, METI, ESRI CHINA (HONG KONG), (C) OPENSTREETMAP CONTRIBUTORS, AND THE GIS USER COMMUNITY
4. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: UTM ZONE 17N

CLIENT
REGIONAL MUNICIPALITY OF NIAGARA

PROJECT
SOUTH NIAGARA FALLS WASTEWATER SOLUTIONS
SCHEDULE C CLASS ENVIRONMENTAL ASSESSMENT

CONSULTANT
GOLDER

YYYY-MM-DD	2020-05-19
DESIGNED	PR
PREPARED	PR
REVIEWED	GVA
APPROVED	GVA

TITLE
PREDICTED EFFLUENT PLUME IN JANUARY

PROJECT NO.	CONTROL	REV.	FIGURE
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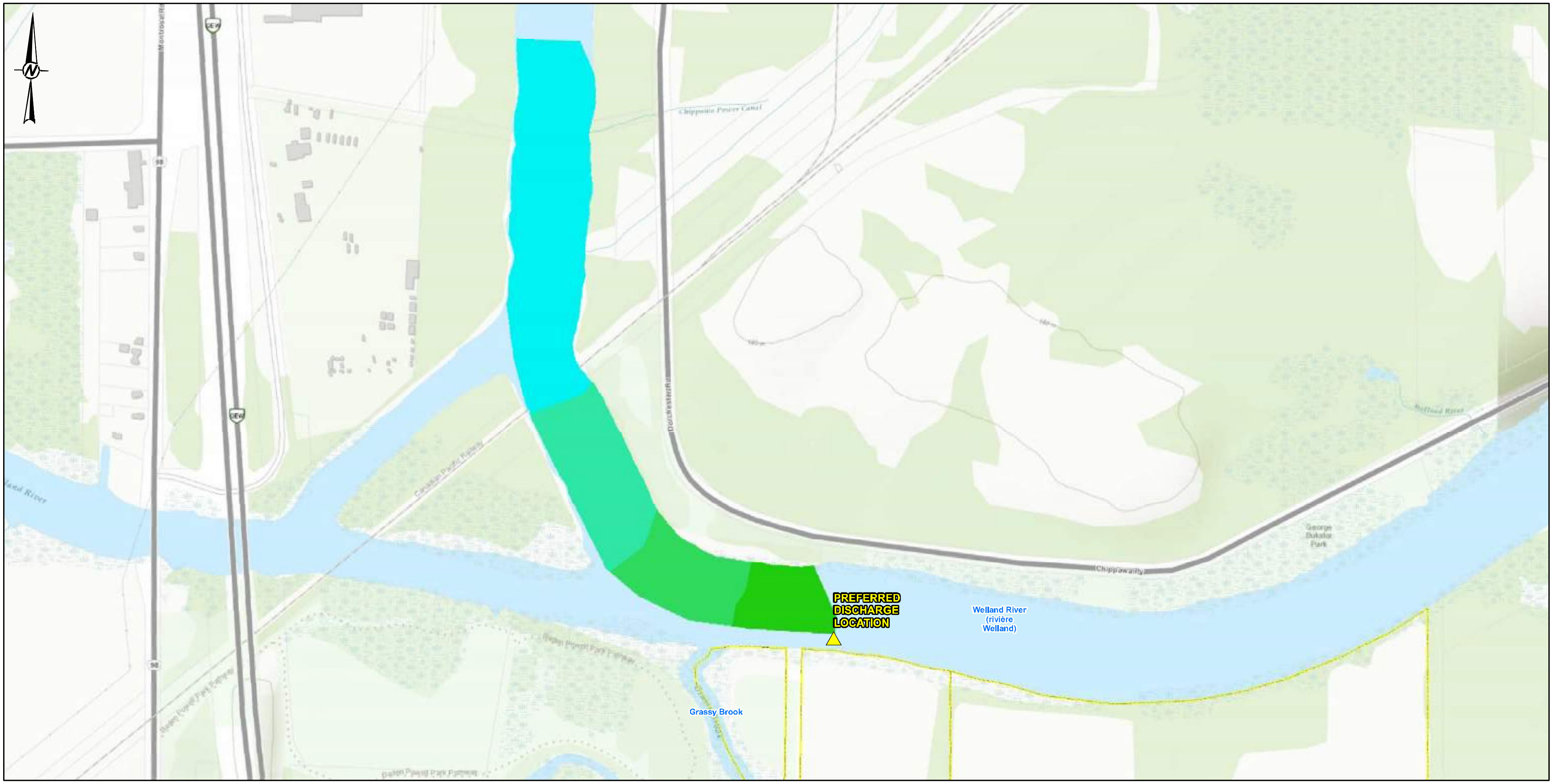


LEGEND

- PREFERRED DISCHARGE LOCATION
- WATERBODY
- PREFERRED SITE LOCATION 8

PLUME DILUTION

- 0 - 100
- 100 - 200
- 200 - 300
- 300 - 400
- 400 - 500
- 500 - 600
- 600- 700

0 100 200
1:5,000 METRES

REFERENCE(S)

1. BASE DATA - MNRF LIO, OBTAINED 2019
2. PRODUCED BY GOLDER ASSOCIATES LTD UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES AND FORESTRY, © QUEENS PRINTER 2019
3. SOURCES: ESRI, HERE, GARMIN, INTERMAP, INCREMENT P CORP., GEBCO, USGS, FAO, NPS, NRCAN, GEOBASE, IGN, KADASTER NL, ORDNANCE SURVEY, ESRI JAPAN, METI, ESRI CHINA (HONG KONG), (C) OPENSTREETMAP CONTRIBUTORS, AND THE GIS USER COMMUNITY
4. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: UTM ZONE 17N

CLIENT
REGIONAL MUNICIPALITY OF NIAGARA

PROJECT
SOUTH NIAGARA FALLS WASTEWATER SOLUTIONS
SCHEDULE C CLASS ENVIRONMENTAL ASSESSMENT

CONSULTANT
GOLDER

YYYY-MM-DD	2020-05-19
DESIGNED	PR
PREPARED	PR
REVIEWED	GVA
APPROVED	GVA

TITLE
PREDICTED EFFLUENT PLUME IN JULY

PROJECT NO.	CONTROL	REV.	FIGURE
18104462	0010	0.0	18

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B
25mm

3.3.6 Sensitivity Analysis

This section provides a sensitivity analysis of the modelling results to effluent flow rate and flow in Chippewa Creek.

Effluent Flow Rate Variations

A sensitivity analysis was performed for the effluent flow rate for the months of January and July by considering effluent flow rates of 20 MLD and 120 MLD. The effects of variations in effluent flow rate are shown on Figure 19.

In January (plume tends to float), the dilution increased for the reduced effluent flow rate and decreased for the elevated effluent flow rate. This suggests that the increased exit velocity at higher flow rates produced a longer and thinner plume when compared to the design flow.

In July (plume tends to sink) there was a small increase in the far-field dilution when the effluent flow rate decreased. However, when the effluent flow rate increased, the near-field dilution increases. This suggests that the increased exit velocity counteracts some of the negative buoyancy of the effluent in July.

In all cases except the high flow January scenario, the plume dilution reaches 200:1 in a distance of less than 350 m. However, because the high flow cases are expected to have a duration of a few hours at most, there are no adverse affects expected from the low effluent dilution expected.

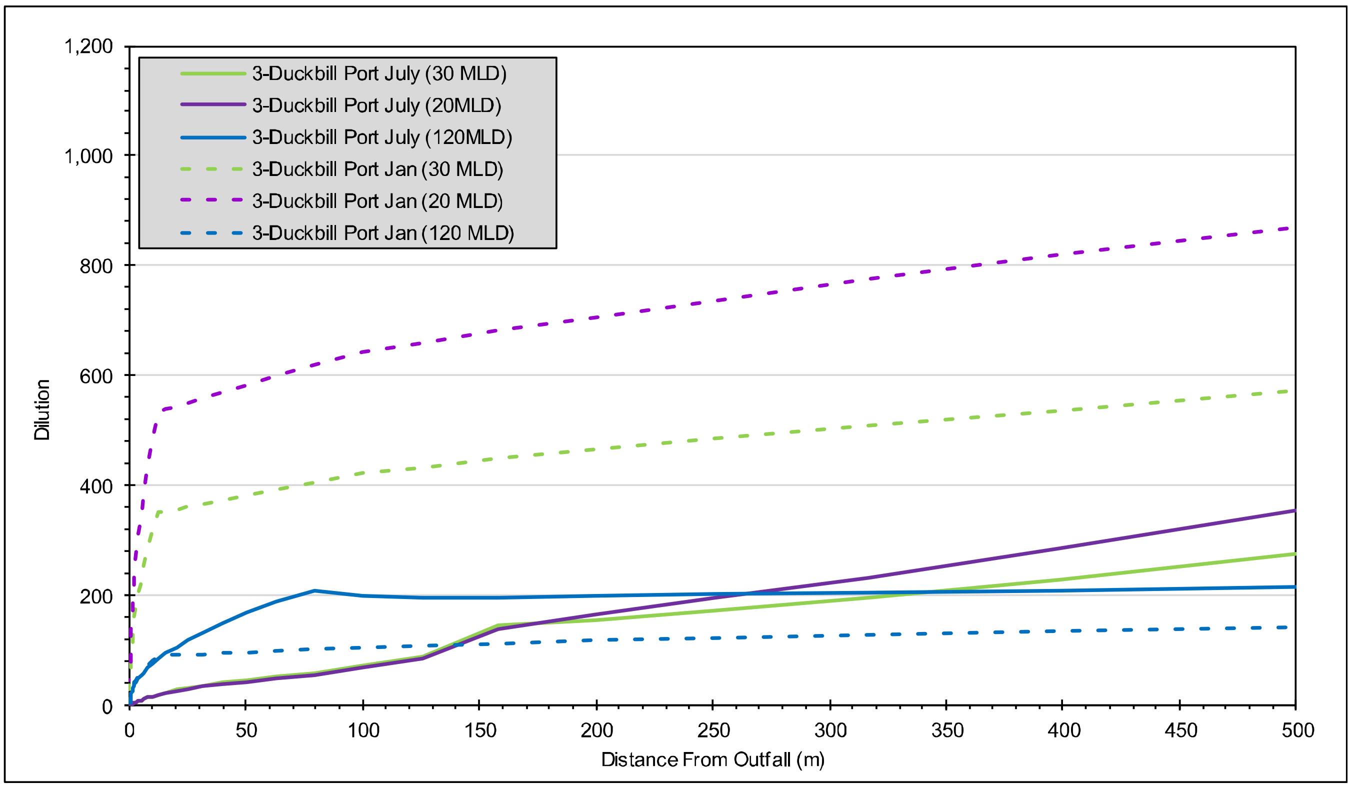


Figure 19: Distance-Dilution Plots for Effluent Flow Rate Variation Sensitivity Analysis

Ambient Flow Rate Variations

Changes in the flow in Chippewa Creek will change the current speed in the area of the outfall and can potentially affect the performance of the outfall. A sensitivity analysis was performed that compared the outfall performance for selected maximum flows from Table 18. July was selected as the highest Chippewa Creek flow (472 m^3/s) when the effluent plume is expected to have a tendency to sink and December was selected as the highest Chippewa Creek flow (468 m^3/s) when the effluent plume is expected to have a tendency to float.

In July and November, the ambient current speeds are expected to increase to 0.49 m/s when the high flow conditions in Chippewa Creek are considered.

The sensitivity analysis for the ambient flow rate variation showed that an increase in the ambient current velocity enhanced the mixing of the effluent, increasing by approximately 30% for December (plume tends to float) in both the near-filed and far-field as shown on Figure 20. However, in July (plume tends to sink) the increased current speeds had no effect in the near field and only a small decrease (10%) of the effluent dilution in the far-field.

The sensitivity analysis suggests that there are no concerns related to outfall performance during high flow conditions in Chippewa Creek. In both cases, the effluent dilution reaches 200:1 at distances similar to the corresponding primary scenarios.

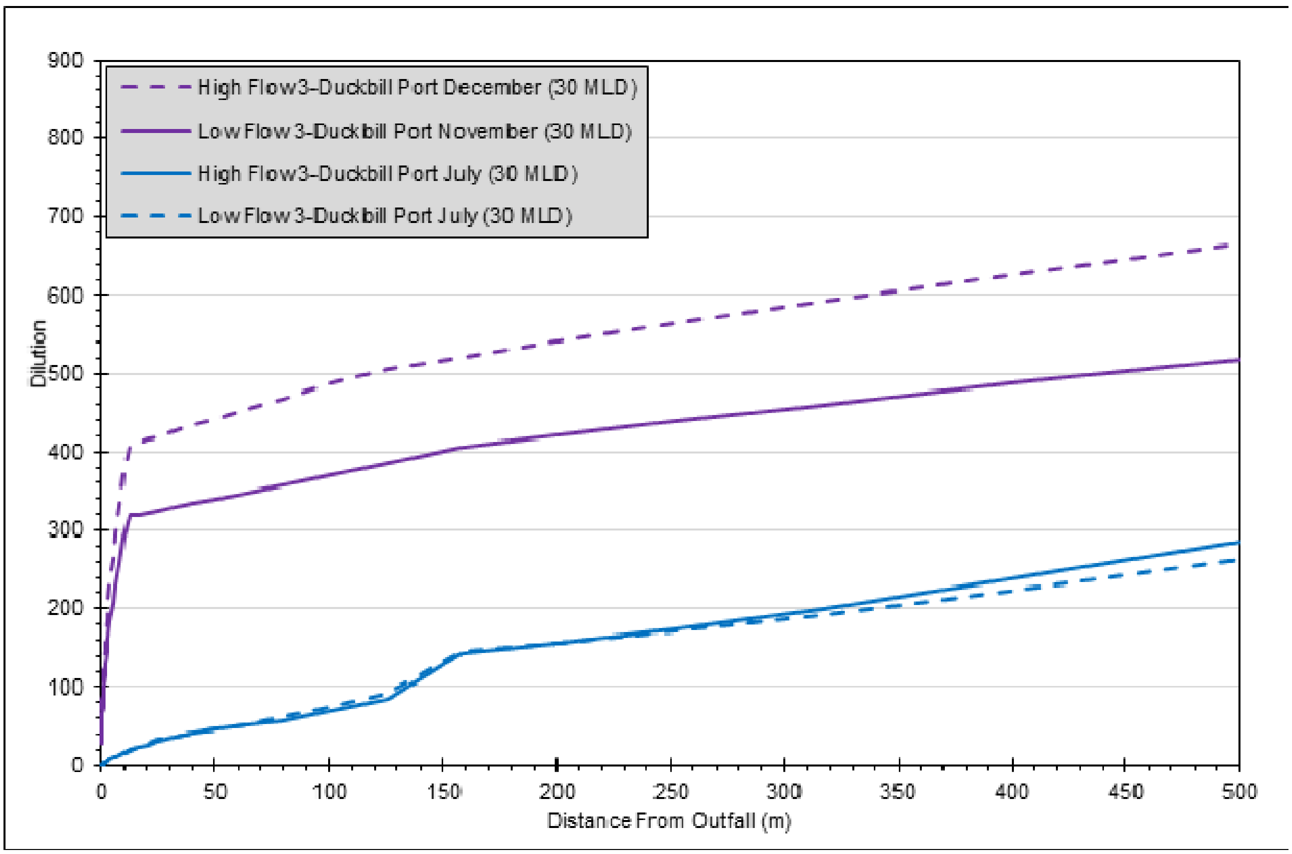


Figure 20: Distance-Dilution Plots for Ambient Flow Rate Variation Sensitivity Analysis

3.3.7 Mixing Zone Assessment Summary

The results of the mixing zone assessment are summarized in the following points;

- The conceptual outfall design that includes duckbill valves provides reasonable performance for most of the scenarios modelled. The only exception is during high effluent flow rates (120 MLD) during the summer when plume dilution does not reach 200:1 within 1,000 m. However, high effluent flow rates are expected to occur infrequently and have a duration of a few hours or less.
- In periods when the plume has a tendency to float (October to May), a 20:1 dilution is expected within 1 m of the outfall.
- In periods when the plume has a tendency to sink (June to September), the sinking jets produce lower dilution factors closer to outfall and the distance to a 20:1 dilution is approximately 13 m
- In periods when the plume has a tendency to sink (June to September), the sinking jets produce lower dilution factors closer to outfall and the distance to a 200:1 dilution is approximately 350 m
- In periods when the plume has a tendency to float (October to May), a 200:1 dilution is expected within 5 m of the outfall.
- Variations in the Chippewa Creek flow are not expected to noticeably affect the performance of the outfall design.
- In general, variations in the effluent flow rate are not expected to adversely affect the performance of the outfall design.

4.0 DERIVATION OF RECOMMENDED EFFLUENT LIMITS

The following sections outline the development of the recommended effluent objectives and limits based on the ACS and include the following details:

- the applicable water quality assessment points;
- if specific parameters meet or exceed relevant criteria and whether a Policy 2 Condition applies;
- the critical months for each parameter; and
- an appropriate treatment technology.

The available assimilative capacity is first considered without the effluent inputs from the new WWTP to determine if there is any capacity in the system for each of the parameters at the local compliance point. In cases where there was assimilative capacity to assimilate effluent, a treatment technology was selected that could meet the maximum allowable effluent concentrations for each parameter. In cases were there was no available assimilative capacity (e.g., Policy 2), the effluent quality was selected such that the effluent concentration would be equal or less than the existing background conditions.

The typical effluent quality for the available treatment technologies considered in this study, based on information available from the MECP (MECP 2019), are summarized in Table 21.

Table 21: Typical Effluent Quality for Various Treatment Processes

Process	Effluent Parameter[1,2]			
	$CBOD_5$ (mg/L)	Total Suspended Solids (mg/L)	Total Phosphorus (mg/L)	Total Ammonia (mg/L as N)[3]
Conventional Activated Sludge System				
Without Phosphorus Removal	25	25	3.5	15 to 20
With Phosphorus Removal	25	25	<1.0	15 to 20
With Phosphorus Removal and Filtration	10	10	0.3	15 to 20
With Nitrification and Phosphorus Removal	25	25	<1.0	<3
Membrane Bioreactor				
Without Phosphorus Removal	2	1	3.0	15 – 20
With Phosphorus Removal	2	1	0.1	15 – 20
With Phosphorus Removal and Filtration	2	1	0.1	0.3

Notes:

1. Taken from "Design Considerations for Sewage Treatment Plants" (MECP 2019)
2. The above values are based on raw sewage with CBOD5 = 150-200 mg/L, Soluble CBOD5 = 50% of CBOD5, TSS = 150-200 mg/L, TP = 6-8 mg/L, TKN = 30-40 mg/L, TAN = 20-25 mg/L.
3. TAN (total ammonia nitrogen) concentrations may be lower during warm weather conditions if nitrification occurs.

With regard to parameters not listed in Table 21, the following assumptions have been used:

- any treatment plant with disinfection can expect to have an *E. coli* concentration objective of less than 200 cfu/100 mL;

- if needed, aeration of the dissolved oxygen concentration in the final effluent can be provided to at least 80% of the saturation concentration; and
- The expected effluent nitrate concentration from an activated sludge system without denitrification was assumed to be 20 mg/L.

The preferred discharge location will release effluent to the Chippewa Creek between Lyons Creek and Triangle Island. The existing water quality in Chippewa Creek is dominated by the water quality in the Niagara River. Under normal conditions, the effluent will travel downstream into the HEPC and eventually enter the Niagara River at the Sir Adam Beck GS. The local compliance point (A3) is in Chippewa Creek just upstream of Triangle Island and the system compliance point (A5) is in the HEPC below the existing Niagara Falls WWTP, so that the combined effects of both plants are considered in the ACS. The preferred discharge location is not expected to affect water quality in Welland River East or in the Niagara River upstream of the Sir Adam Beck GS.

In typical assimilative capacity assessments, it is expected that the low-flow conditions (e.g., worst case conditions) will result in the most restrictive conditions and the results from GoldSim and the mass balance modelling should be similar. In this assessment there are many cases where GoldSim predicts maximum allowable effluent concentrations that are lower than those predicted by the mass balance modelling. The differences occur because the flow conditions in the various inflows are independent and low-flow conditions do not necessarily occur at the same time for the different inflows (e.g., a high flow event after a rainfall event in the Welland River East at the same time as a low flow occurs in the HEPC due to the operation of the ICD). A review of the modelling results suggests that high flow events in the Welland River East occurring at the same time as low HEPC flows can alter the maximum allowable effluent concentrations in two ways:

1) Because the water quality in the Welland River East is degraded, the higher relative contribution of water into the from the river reduces the assimilative capacity at the system compliance point (below existing Niagara Falls WWTP).
2) Because the flow in Chippewa Creek is assumed to be the difference between the flow in the HEPC and the flow entering from the Welland River East, a high flow event in the river will cause a decrease in the Chippewa Creek flow and reduce the amount of water available for dilution.

The following sections outline the rationales for developing the proposed effluent limits based on existing conditions, results from all the modelling, specific MECP end-of-pipe toxicity limits, and the typical effluent quality from the available treatment technologies.

4.1 Total Phosphorus

The measured monthly 75th percentile total phosphorus concentrations in Chippewa Creek range from 0.021 mg/L (July and September) to 0.49 mg/L (December) and are effectively the same as the measured conditions in the Niagara River. There are additional constraints at the system compliance point caused by the discharge of effluent into the HEPC from the existing Niagara Falls WWTP and by the mass contribution from the Welland River East which exceeds objectives year-round.

The calculated maximum allowable effluent concentration for total phosphorus at the local and system compliance points and regulatory objectives are presented in Table 22.

Table 22: Maximum Allowable Monthly Total Phosphorus Concentrations for Discharge to Chippewa Creek

Month	GoldSim Modelling[1]		Mass-Balance Modelling[2]	
	Local Compliance Point (mg/L)	System Compliance Point (mg/L)	Local Compliance Point (mg/L)	System Compliance Point (mg/L)
January	nc	nc	nc	nc
February	nc	nc	nc	nc
March	2.7	nc	4.9	nc
April	2.2	nc	3.8	nc
May	3.2	nc	3.8	nc
June	7.6	nc	7.7	0.4
July	9.9	6.5	9.9	6.5
August	8.2	5.4	8.9	6.2
September	8.1	3.9	8.9	6.0
October	3.3	nc	5.3	0.1
November	nc	nc	nc	nc
December	nc	nc	nc	nc

Notes:

1. Calculated concentrations based on allowable mass capacity based on 5% probability of no exceedance.
2. Calculated based on low flow conditions occurring for all inflows simultaneously.
3. 'nc' denotes no assimilative capacity at compliance point.
4. Modelled results exclude effects of CSOs and WWTP bypasses.

The elevated total phosphorus concentrations under baseline conditions result in Policy 2 conditions at the local compliance point in the November to February. At the local compliance point, Chippewa Creek can accept total phosphorus concentration of 2.2 mg/L or greater in the effluent in all months except for November to February. At the system compliance point, elevated phosphorus concentrations under baseline conditions are experienced from October to June due to inputs from the Welland River East and existing Niagara Falls WWTP.

An effluent limit for total phosphorus of 0.75 mg/L is recommended based in the following rationale:

- On an annual basis, there is sufficient capacity to accept an effluent concentration greater than 0.5 mg/L.
- The effluent flow rate represents less than 0.1% of the total flow in Chippewa Creek and as such the contributions of the proposed discharge will cause negligible increase in the total phosphorus concentrations within Chippewa Creek and the HEPC.
- The elevated phosphorus concentration in Chippewa Creek are only experienced during October to February, which is outside the algae growing season. Furthermore, the elevated background phosphorus concentrations are the result of factors outside the study area (e.g., inflow from the Niagara River and Lyons Creek).
- Similarly, the effluent flow rate is insignificant when compared to the flow in the Niagara River below the Sir Adam Beck GS.

The predicted plume centreline concentration for January and July are provided on Figure 21 for an effluent discharge rate of 30 MLD. In January, while the plume is never expected to meet the PWQO for total phosphorus (0.03 mg/L) due to elevated background conditions, the plume is expected to be within 0.003 mg/L of the ambient within 10 m of the outfall. In July, the plume is expected to meet the PWQO at a downstream distance of approximately 125 m.

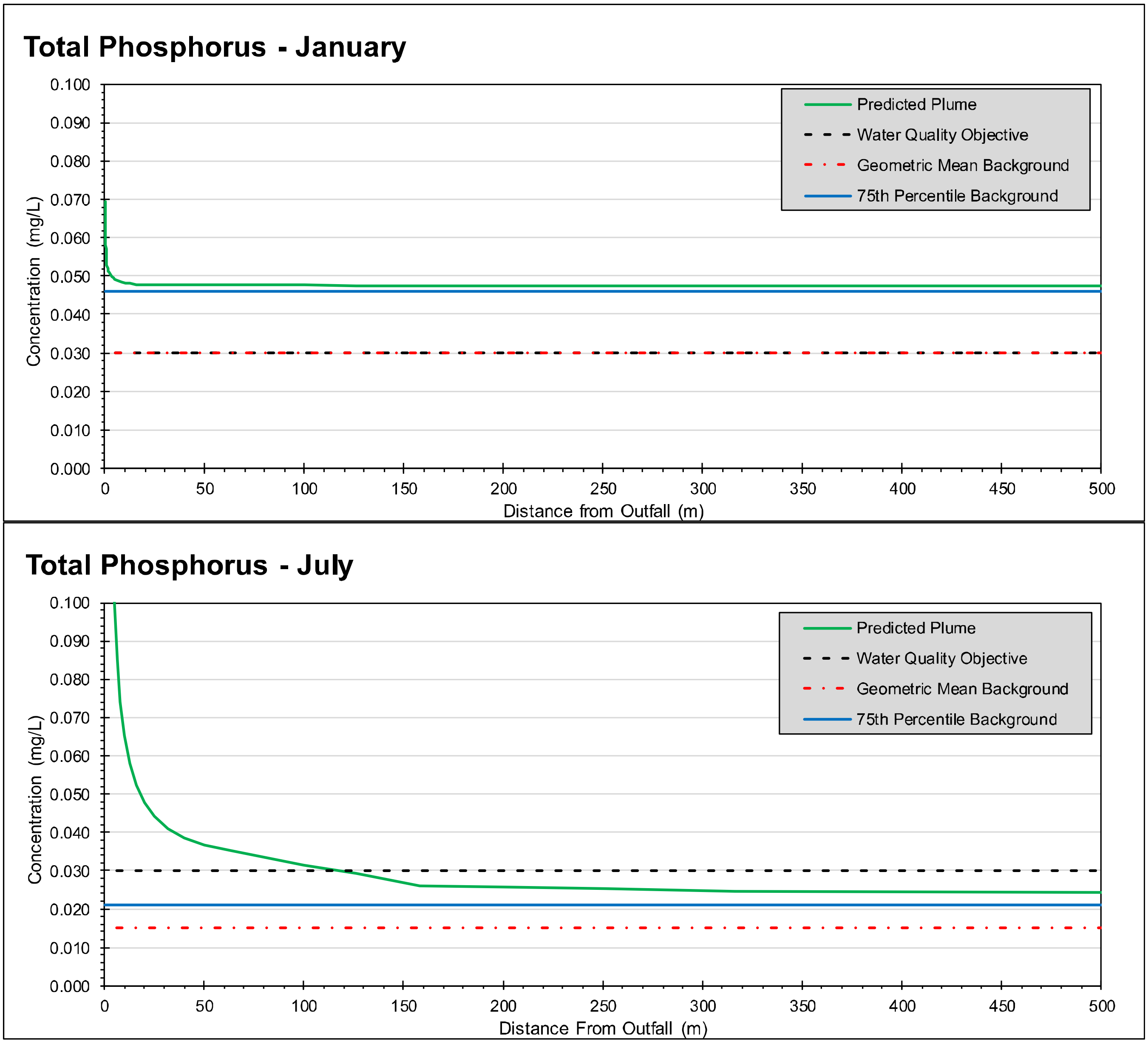


Figure 21: Predicted Total Phosphorus Concentrations Downstream of Outfall

The predicted effects of the proposed WWTP on the monthly total phosphorus concentrations in the receiving waters are summarized and compared to the predicted worst–case existing conditions in Table 23. The existing conditions are predicted using the mass balance model for low-flow conditions and assume that the existing Niagara Falls WWTP is operating at the rated capacity (68.3 MLD) and discharging effluent with a total phosphorus concentration equal to the ECA limits (0.75 mg/L).

The total phosphorus concentrations are expected to increase by approximately 0.0007 mg/L (0.7 µg/L) in Chippewa Creek and the HEPC, an increase of 3.2% or less. In the Niagara River, the increase in total phosphorous are predicted to be approximately 0.0001 mg/L (0.1 µg/L) which represents an increase of 0.3% or less.

Table 23: Summary of Predicted Effects of Project on Total Phosphorus Concentrations

Month	Chippewa Creek at Triangle Island (A3)			HEPC at Montrose Gate (A2)			HEPC Above SAB[1] (A5)			Niagara River Below SAB[1] (A6)		
	Existing (mg/L)	Future (mg/L)	Difference (mg/L)	Existing (mg/L)	Future (mg/L)	Difference (mg/L)	Existing (mg/L)	Future (mg/L)	Difference (mg/L)	Existing (mg/L)	Future (mg/L)	Difference (mg/L)
January	0.0461	0.0468	0.0007 (1.5%)	0.0489	0.0496	0.0007 (1.3%)	0.0504	0.0511	0.0007 (1.3%)	0.0464	0.0465	0.0001 (0.1%)
February	0.0311	0.0318	0.0007 (2.3%)	0.0335	0.0342	0.0007 (2.0%)	0.0351	0.0358	0.0007 (1.9%)	0.0314	0.0314	0.0001 (0.2%)
March	0.0252	0.0259	0.0007 (2.9%)	0.0313	0.0320	0.0007 (2.2%)	0.0329	0.0336	0.0007 (2.1%)	0.0258	0.0259	0.0001 (0.2%)
April	0.0264	0.0270	0.0007 (2.6%)	0.0308	0.0314	0.0007 (2.2%)	0.0323	0.0329	0.0007 (2.0%)	0.0267	0.0268	0.0001 (0.2%)
May	0.0264	0.0271	0.0007 (2.6%)	0.0290	0.0297	0.0007 (2.3%)	0.0305	0.0312	0.0007 (2.2%)	0.0266	0.0267	0.0001 (0.2%)
June	0.0226	0.0233	0.0007 (3.1%)	0.0281	0.0288	0.0007 (2.4%)	0.0296	0.0303	0.0007 (2.2%)	0.0231	0.0231	0.0001 (0.2%)
July	0.0208	0.0215	0.0007 (3.2%)	0.0228	0.0234	0.0006 (2.8%)	0.0243	0.0249	0.0006 (2.7%)	0.0210	0.0211	0.0001 (0.3%)
August	0.0217	0.0224	0.0007 (3.1%)	0.0230	0.0236	0.0007 (2.9%)	0.0245	0.0251	0.0007 (2.7%)	0.0219	0.0219	0.0001 (0.2%)
September	0.0215	0.0222	0.0007 (3.2%)	0.0230	0.0237	0.0007 (2.9%)	0.0246	0.0252	0.0007 (2.7%)	0.0217	0.0218	0.0001 (0.3%)
October	0.0249	0.0256	0.0007 (2.8%)	0.0284	0.0291	0.0007 (2.4%)	0.0300	0.0307	0.0007 (2.3%)	0.0252	0.0253	0.0001 (0.2%)
November	0.0333	0.0340	0.0007 (2.2%)	0.0372	0.0378	0.0007 (1.9%)	0.0387	0.0394	0.0007 (1.8%)	0.0337	0.0337	0.0001 (0.2%)
December	0.0492	0.0499	0.0007 (1.4%)	0.0526	0.0533	0.0007 (1.3%)	0.0542	0.0548	0.0007 (1.2%)	0.0496	0.0497	0.0001 (0.1%)
Annual	0.0290	0.0297	0.0007 (2.4%)	0.0322	0.0329	0.0007 (2.1%)	0.0338	0.0344	0.0007 (2.0%)	0.0291	0.0292	0.0001 (0.2%)

Notes:

1. SAB – Sir Adam Beck GS

4.2 Nitrate

The measured 75th percentile nitrate concentrations in Chippewa Creek range from 0.16 mg/L (September) to 0.54 mg/L (February) and are effectively the same as the measured conditions in the Niagara River. The modelled baseline concentrations and calculated maximum allowable effluent concentration for nitrate at the local and system compliance points and regulatory objectives are presented in Table 24.

At the local and system compliance points, nitrate concentrations are below the regulatory objectives for each month. In general, the local compliance point provides the most restrictive conditions. Based on the modelling results, both the local and system compliance points can accept effluent nitrate concentrations in excess of 2,500 mg/L.

Based on the assumptions in Section 4.0, a conventional activated sludge system without denitrification is expected to provide effluent nitrate concentrations of 20 mg/L. As a result, nitrate limits would not be required for the proposed discharge location to Chippewa Creek.

Table 24: Maximum Allowable Monthly Nitrate Concentrations for Discharge to Chippewa Creek

Month	GoldSim Modelling[1]		Mass-Balance Modelling[2]	
	Local Compliance Point (mg/L)	System Compliance Point (mg/L)	Local Compliance Point (mg/L)	System Compliance Point (mg/L)
January	2,530	2,594	2,772	2,777
February	2,186	2,302	2,744	2,751
March	2,457	2,687	2,716	2,762
April	2,670	2,816	2,825	2,887
May	2,725	2,851	2,799	2,863
June	2,910	2,965	2,785	2,828
July	3,025	3,139	2,997	3,091
August	2,933	3,077	3,024	3,122
September	2,837	3,010	2,974	3,064
October	2,702	2,840	2,890	2,939
November	**2,553**	2,704	2,752	2,791
December	2,484	2,602	2,661	2,687

Notes:

1. Calculated concentrations based on allowable mass capacity based on 5% probability of no exceedance.
2. Calculated based on low flow conditions occurring for all inflows simultaneously.
3. Modelled results exclude effects of CSOs and WWTP bypasses.

The predicted plume centreline concentration for January and July are provided on Figure 22 for an effluent discharge rate of 30 MLD. In January and July, the plume is expected to meet the CCME guideline (3 mg/L) within a downstream distance of approximately 10 m.

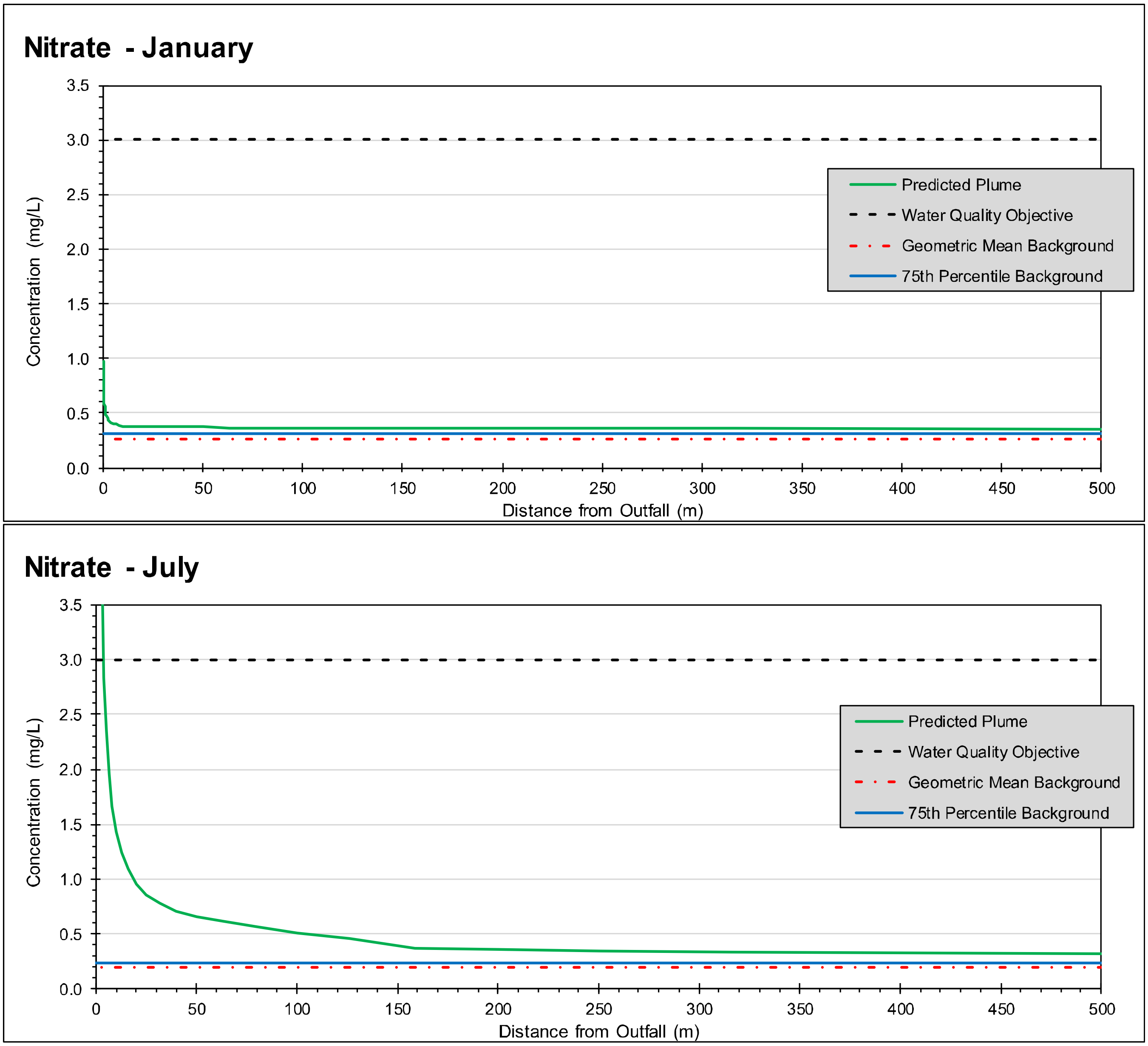


Figure 22: Predicted Nitrate Concentrations Downstream of Outfall

The predicted effects of the Project on the monthly nitrate concentrations in the study are summarized and compared to the predicted worst-case existing conditions in Table 25. The existing conditions are predicted using the mass balance model for low-flow conditions and assume that the existing Niagara Falls WWTP is operating at the rated capacity (68.3 MLD) and discharging effluent with a nitrate concentration equal to the monthly 75th percentile of the measured effluent data (8.41 to 9.71 mg/L).

The nitrate concentrations are expected to increase by approximately 0.02 mg/L in Chippewa Creek and the HEPC, an increase of 11.5% or less. In the Niagara River, the increases in nitrate concentrations are predicted to be less than 0.002 mg/L which represents an increase of 0.9% or less.

Table 25: Summary of Predicted Effects of Project on Nitrate Concentrations

Month	Chippewa Creek at Triangle Island (A3)			HEPC at Montrose Gate (A2)			HEPC Above SAB (A5)			Niagara River Below SAB (A6)		
	Existing (mg/L)	Future (mg/L)	Difference (mg/L)	Existing (mg/L)	Future (mg/L)	Difference (mg/L)	Existing (mg/L)	Future (mg/L)	Difference (mg/L)	Existing (mg/L)	Future (mg/L)	Difference (mg/L)
January	0.317	0.336	0.019 (6.0%)	0.387	0.405	0.018 (4.7%)	0.407	0.425	0.018 (4.5%)	0.325	0.327	0.002 (0.5%)
February	0.305	0.325	0.019 (6.3%)	0.370	0.388	0.019 (5.1%)	0.388	0.407	0.019 (4.8%)	0.313	0.315	0.002 (0.6%)
March	0.290	0.310	0.020 (6.8%)	0.330	0.349	0.019 (5.7%)	0.348	0.367	0.019 (5.4%)	0.295	0.297	0.002 (0.6%)
April	0.297	0.316	0.019 (6.4%)	0.331	0.349	0.018 (5.5%)	0.348	0.366	0.018 (5.2%)	0.301	0.302	0.001 (0.5%)
May	0.324	0.343	0.019 (5.8%)	0.344	0.362	0.018 (5.3%)	0.363	0.381	0.018 (5.0%)	0.327	0.329	0.001 (0.4%)
June	0.323	0.342	0.019 (5.9%)	0.376	0.394	0.018 (4.8%)	0.390	0.409	0.018 (4.6%)	0.328	0.329	0.001 (0.4%)
July	0.229	0.248	0.018 (8.0%)	0.239	0.257	0.018 (7.4%)	0.254	0.271	0.018 (6.9%)	0.231	0.233	0.001 (0.6%)
August	0.168	0.186	0.019 (11.1%)	0.174	0.191	0.018 (10.3%)	0.189	0.207	0.018 (9.4%)	0.169	0.171	0.001 (0.9%)
September	0.164	0.183	0.019 (11.5%)	0.176	0.194	0.018 (10.4%)	0.192	0.211	0.018 (9.4%)	0.166	0.168	0.002 (0.9%)
October	0.179	0.198	0.019 (10.8%)	0.230	0.249	0.019 (8.1%)	0.246	0.264	0.019 (7.5%)	0.184	0.186	0.002 (0.8%)
November	0.221	0.241	0.020 (9.0%)	0.285	0.304	0.019 (6.7%)	0.302	0.321	0.019 (6.3%)	0.228	0.229	0.002 (0.7%)
December	0.296	0.316	0.020 (6.8%)	0.369	0.388	0.019 (5.2%)	0.390	0.409	0.019 (4.9%)	0.304	0.305	0.002 (0.5%)
Annual	0.259	0.278	0.019 (7.4%)	0.299	0.318	0.018 (6.1%)	0.317	0.335	0.018 (5.8%)	0.263	0.265	0.002 (0.6%)

Notes:

1. SAB – Sir Adam Beck GS

4.3 Ammonia

The effluent limits are typically expressed as total ammonia but are based on the regulatory limit for un-ionized ammonia (chronic toxicity limit of 0.0164 mg/L as N). The fraction of the total ammonia that is unionized is directly related to the water temperature and pH. As such, water temperature and pH for the Niagara River are also described in this section.

The measured 75th percentile ammonia concentrations in Chippewa Creek ranged from 0.012 mg/L (February) to 0.058 mg/L (April) and are effectively the same as the measured conditions in the Niagara River. In the Niagara River, the measured 75th percentile water temperatures ranged from 0.3ºC (February) to 24.5ºC (July) and the measured pH 75th percentile values ranged from 8.10 to 8.40 (no pattern observed). The observed ammonia concentrations and calculated unionized ammonia for the Niagara River are summarized in Table 26.

Table 26: Monthly Observed 75th Percentile Values for Water Temperature, pH, Total Ammonia, and Calculated Unionized Ammonia for Niagara River

Month	Water Temperature[1] (ºC)	pH[1]	Unionized Ammonia Fraction	Total Ammonia[1] (mg/L)	Unionized Ammonia[2] (mg/L)
January	0.7	8.1	1.1%	0.014	0.00015
February	0.3	8.1	1.0%	0.012	0.00013
March	2.5	8.1	1.3%	0.023	0.00029
April	7.8	8.1	1.9%	0.058	0.00112
May	14.1	8.2	3.9%	0.049	0.00190
June	20.3	8.3	7.5%	0.049	0.00366
July	24.5	8.4	12.0%	0.043	0.00512
August	24.4	8.4	12.0%	0.044	0.00529
September	22.5	8.3	8.7%	0.041	0.00355
October	17.5	8.3	6.2%	0.035	0.00216
November	10.1	8.2	2.9%	0.023	0.00067
December	5.2	8.1	1.6%	0.016	0.00025

Notes:

1. values presented represent 75th percentile value of measured data.
2. estimated using equations presented in MOEE (1994).

The monthly maximum allowable effluent concentrations for total ammonia were calculated from the monthly allowable concentrations of unionized ammonia using the monthly measured 75th percentiles of water temperature and pH. The modelled baseline concentrations and calculated maximum allowable effluent concentration for ammonia at the local and system compliance points and estimated monthly regulatory limits are presented in Table 27.

In addition to the maximum allowable effluent concentrations for total ammonia based on assimilative capacity, Table 28 provides the estimated monthly effluent limit based on the end-of-pipe acute toxicity limit of 0.1 mg/L for unionized ammonia and the measured 75th percentile effluent temperatures and pH from the existing Niagara Falls WWTP.

Table 27: Maximum Allowable Monthly Total Ammonia Concentrations for Discharge to Chippewa Creek Based on Water Quality Modelling

Month	GoldSim Modelling[1]		Mass-Balance Modelling[2]	
	Local Compliance Point (mg/L)	System Compliance Point (mg/L)	Local Compliance Point (mg/L)	System Compliance Point (mg/L)
January	1,467	1,554	1,510	1,518
February	855	919	1,542	1,564
March	617	546	931	913
April	361	361	658	663
May	174	154	381	370
June	95	66	172	155
July	102	80	98	78
August	151	135	95	81
September	225	213	151	133
October	512	529	230	216
November	940	1,016	528	525
December	947	1,002	990	997

Notes:

1. Calculated concentrations based on allowable mass capacity based on 5% probability of no exceedance.
2. Calculated based on low flow conditions occurring for all inflows simultaneously.
3. Modelled results exclude effects of CSOs and WWTP bypasses.

Table 28: Maximum Allowable Monthly Total Ammonia Concentrations for Discharge to Chippewa Creek Based on Acute Toxicity of Unionized Ammonia

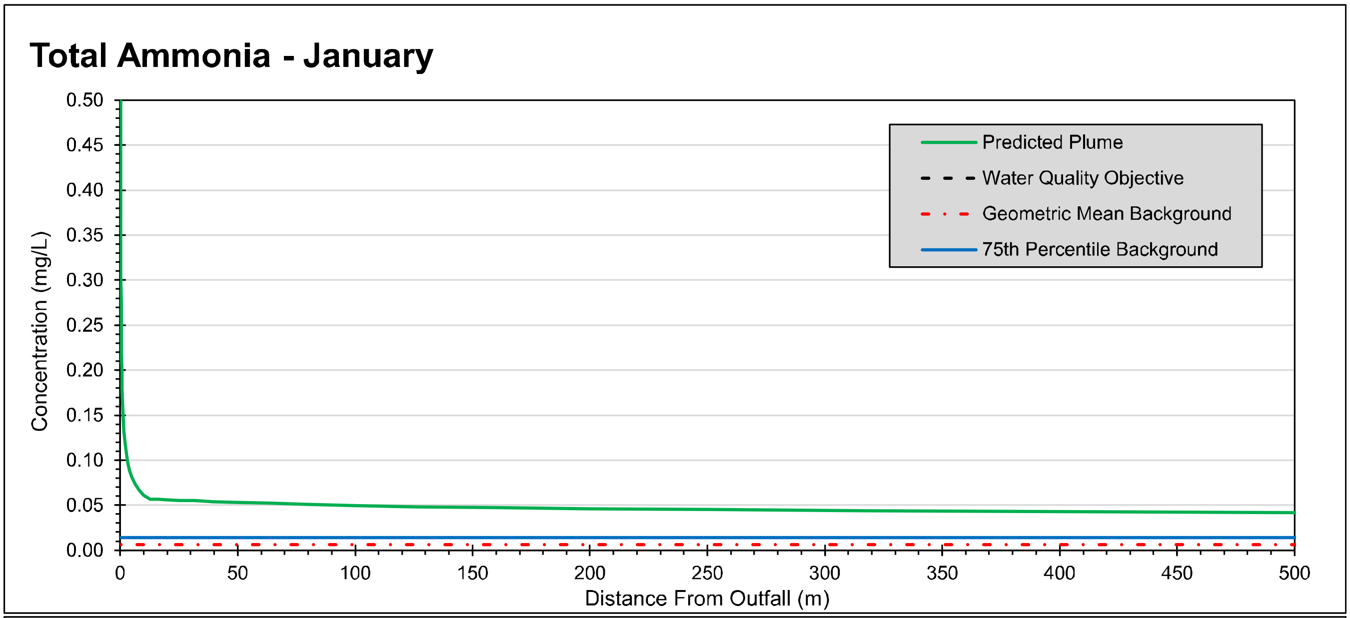
Month	75th Percentile Effluent Temperature[1] (ºC)	75th Percentile Effluent pH[1]	Maximum Allowable Effluent Concentration[2] (mg/L)
January	10.9	7.38	21.1
February	10.9	7.33	23.6
March	11.0	7.38	20.8
April	12.8	7.47	14.9
May	16.6	7.40	13.0
June	19.5	7.40	10.5
July	22.0	7.40	8.80
August	22.7	7.30	10.5
September	22.3	7.34	9.95
October	19.2	7.30	13.5
November	15.5	7.32	16.9
December	13.7	7.35	18.2

Notes:

1. Based on measured effluent temperatures and pH from the existing Niagara Falls WWTP (2015 to 2018).
2. Estimated using equations presented in MOEE (1994).

The predicted maximum allowable total ammonia concentrations based on the assimilative capacity are consistently greater than the values based on the acute toxicity guideline for unionized ammonia. As such, it is recommended that the effluent objectives for total ammonia be based on meeting the acute toxicity limit for unionized ammonia at end-of-pipe and monthly water temperature and pH. Based on the resulting values presented in Table 28, the recommended total ammonia objectives are recommended to be 8.8 mg/L from May to October and 15.0 mg/L from November to April.

The predicted plume centreline concentrations for January and July are provided on Figure 23 for an effluent discharge rate of 30 MLD. The total ammonia guidelines for January and July are 1.51 mg/L and 0.14 mg/L respectively based on monthly water temperatures and pH. The guideline for total ammonia for January is not shown on Figure 23 since it is greater than the predicted and measured concentrations shown on the figure. In January and July, the plume is expected to meet the monthly PWQO guideline within a downstream distance of approximately 130 m.



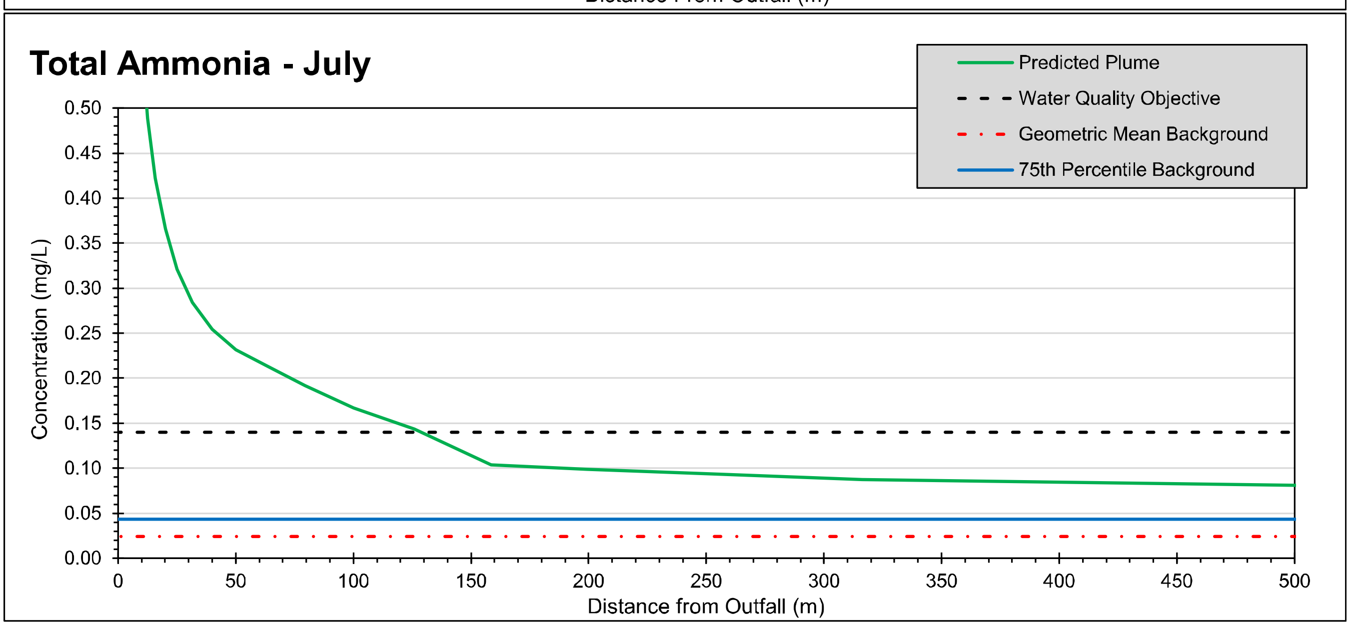


Figure 23: Predicted Total Ammonia Concentrations Downstream of Outfall

The predicted effects of the Project on the monthly total ammonia concentrations in the study are summarized and compared to the predicted worst-case existing conditions in Table 29. The existing conditions are predicted using the mass balance model for low-flow conditions and assume that the existing Niagara Falls WWTP is operating at the rated capacity (68.3 MLD) and discharging effluent with a total ammonia concentration equal to the monthly 75th percentile of the measured effluent data (6.23 to 10.0 mg/L).

The total ammonia concentrations are expected to increase by approximately 0.05 mg/L in Chippewa Creek and the HEPC. In the Niagara River, the increases in total ammonia concentrations are predicted to be approximately 0.001 mg/L which represents an increase of 9% or less.

Table 29: Summary of Predicted Effects of Project on Total Ammonia Concentrations

Month	Chippewa Creek at Triangle Island (A3)			HEPC at Montrose Gate (A2)			HEPC Above SAB (A5)			Niagara River Below SAB (A6)		
	Existing (mg/L)	Future (mg/L)	Difference (mg/L)	Existing (mg/L)	Future (mg/L)	Difference (mg/L)	Existing (mg/L)	Future (mg/L)	Difference (mg/L)	Existing (mg/L)	Future (mg/L)	Difference (mg/L)
January	0.014	0.029	0.015 (103.5%)	0.036	0.050	0.014 (39.0%)	0.057	0.071	0.014 (24.4%)	0.018	0.019	0.001 (7.2%)
February	0.012	0.027	0.015 (119.4%)	0.022	0.036	0.014 (64.9%)	0.043	0.057	0.014 (33.3%)	0.015	0.016	0.001 (8.8%)
March	0.023	0.038	0.015 (63.9%)	0.056	0.071	0.014 (25.5%)	0.075	0.090	0.014 (19.0%)	0.028	0.029	0.001 (4.5%)
April	0.058	0.072	0.014 (24.6%)	0.066	0.080	0.014 (20.8%)	0.079	0.093	0.014 (17.3%)	0.060	0.061	0.001 (1.9%)
May	0.049	0.057	0.008 (17.1%)	0.057	0.065	0.008 (14.1%)	0.072	0.080	0.008 (11.1%)	0.051	0.051	0.001 (1.3%)
June	0.049	0.057	0.008 (17.2%)	0.054	0.062	0.008 (15.0%)	0.071	0.079	0.008 (11.3%)	0.051	0.051	0.001 (1.2%)
July	0.043	0.051	0.008 (19.0%)	0.044	0.052	0.008 (17.5%)	0.063	0.071	0.008 (12.3%)	0.044	0.045	0.001 (1.4%)
August	0.044	0.052	0.008 (18.6%)	0.045	0.053	0.008 (17.6%)	0.060	0.068	0.008 (13.1%)	0.045	0.046	0.001 (1.4%)
September	0.041	0.049	0.008 (20.4%)	0.047	0.055	0.008 (17.1%)	0.063	0.071	0.008 (12.6%)	0.043	0.044	0.001 (1.6%)
October	0.035	0.044	0.009 (24.4%)	0.039	0.048	0.008 (20.9%)	0.058	0.066	0.008 (14.2%)	0.037	0.038	0.001 (1.8%)
November	0.023	0.038	0.015 (64.9%)	0.032	0.046	0.015 (45.7%)	0.049	0.063	0.014 (29.6%)	0.025	0.027	0.001 (4.7%)
December	0.016	0.031	0.015 (95.0%)	0.034	0.049	0.015 (42.4%)	0.053	0.068	0.015 (27.3%)	0.019	0.020	0.001 (6.2%)
Annual	0.034	0.046	0.011 (33.4%)	0.045	0.056	0.011 (24.7%)	0.062	0.073	0.011 (17.7%)	0.037	0.038	0.001 (2.5%)

Notes:

1. SAB – Sir Adam Beck GS

4.4 *E. coli*

The measured 75th percentile *E. coli* concentrations in the Niagara River at the drinking water intake range from 3 cfu/100ml (May) to 11 cfu/100ml (January). There are constraints at the system compliance point caused by the discharge of effluent into the HEPC from the existing Niagara Falls WWTP and by the contribution from the Welland River East which exceeds objectives year-round. The calculated maximum allowable effluent concentration for total phosphorus at the local and system compliance points and regulatory objectives are presented in Table 30.

There are limitations on the discharge at the system compliance point from November to March and in September due to contributions from Welland River East. As such, the effluent concentration is not to exceed background conditions in the HEPC. The measured *E. coli* concentrations in the HEPC range from 5 to over 16,000 cfu/ 100 mL with an average of over 1,600 cfu/100 mL.

An effluent limit for *E. coli* of 200 cfu/100 mL is recommended and is consistent with other treatment plants in the area and recognizes that the HEPC is not used for body-contact recreation. This value is also well below the measured *E. coli* concentrations in the HEPC.

Table 30: Maximum Allowable Monthly *E. coli* Concentrations for Discharge to Chippewa Creek

Month	GoldSim Modelling[1]		Mass-Balance Modelling[2]	
	Local Compliance Point (cfu/100 mL)	System Compliance Point (cfu/100 mL)	Local Compliance Point (cfu/100 mL)	System Compliance Point (cfu/100 mL)
January	79,518	nc	91,711	nc
February	75,315	nc	91,467	72,188
March	86,722	nc	95,937	822
April	92,226	67,543	97,791	89,654
May	98,596	97,296	101,149	100,264
June	103,967	97,529	99,801	94,869
July	104,734	102,999	103,824	102,346
August	98,330	95,695	101,943	100,079
September	90,825	nc	95,634	31,610
October	85,688	10,447	92,334	71,848
November	85,288	nc	92,034	2,717
December	84,521	nc	90,788	nc

Notes:

1. Calculated concentrations based on allowable mass capacity based on 5% probability of no exceedance.
2. Calculated based on low flow conditions occurring for all inflows simultaneously.
3. Modelled results exclude effects of CSOs and WWTP bypasses.

The predicted plume centreline concentrations for January and July are provided on Figure 24 for an effluent discharge rate of 30 MLD. In January and July, the plume is expected to meet the PWQO (100 cfu/100 mL) within a downstream distance of approximately 10 m. The PWQO is not shown on Figure 24 since the measured and predicted concentrations near the outfall are well below the PWQO.

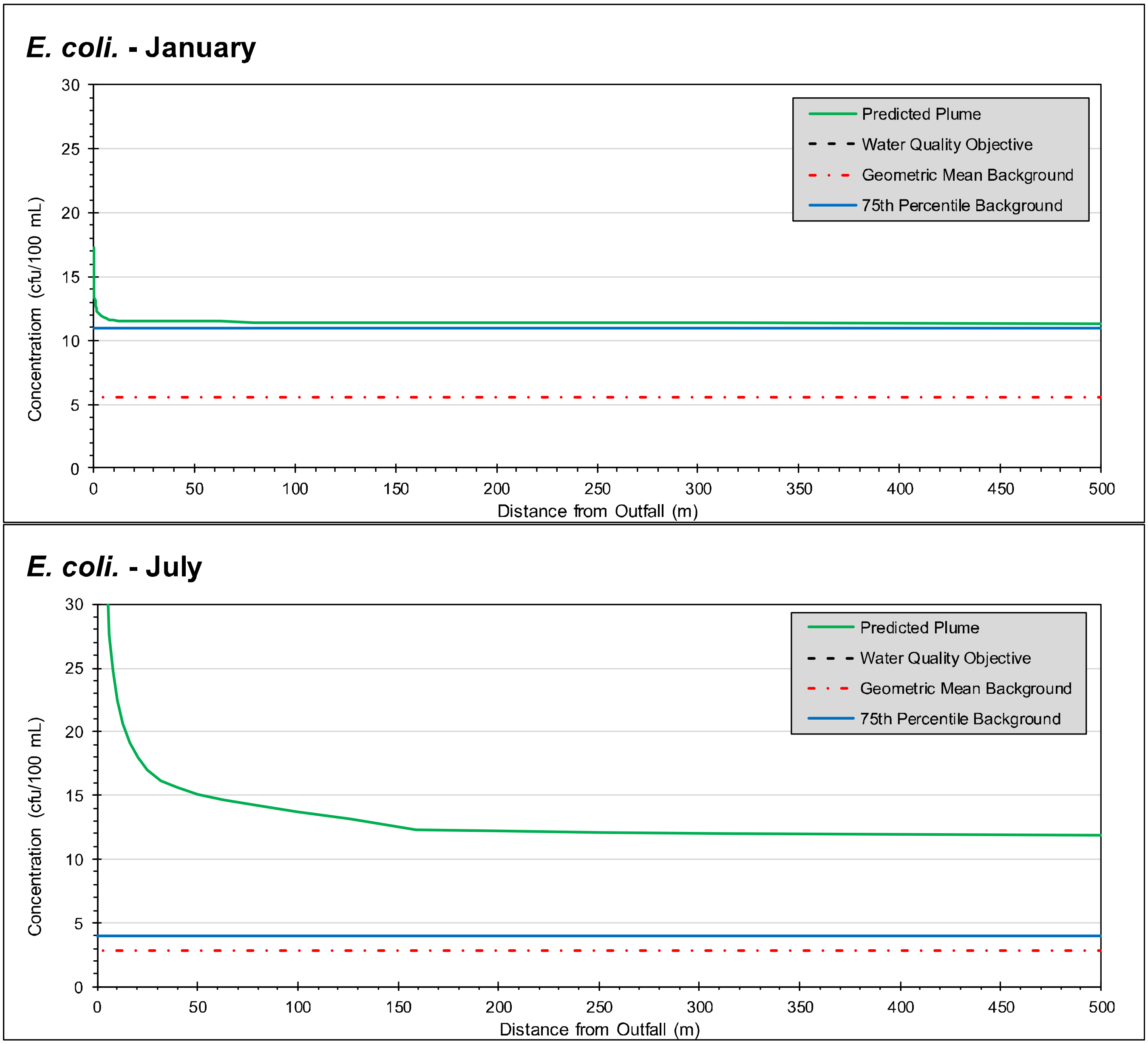


Figure 24: Predicted *E. coli* Concentrations Downstream of Outfall

The predicted effects of the Project on the monthly *E coli* concentrations in the study are summarized and compared to the predicted worst-case existing conditions in Table 30. The existing conditions are predicted using the mass balance model for low-flow conditions and assume that the existing Niagara Falls WWTP is operating at the rated capacity (68.3 MLD) and discharging effluent with an *E. coli* concentration equal to the ECA limits (200 cfu/100 mL).

The *E. coli* concentrations are expected to increase by approximately 0.2 cfu/100 mL in Chippewa Creek and the HEPC, an increase of 6% or less. In the Niagara River, the increases in *E. coli* concentrations are predicted to be less than 0.01 cfu/100 mL which represents an increase of 0.4% or less.

Table 31: Summary of Predicted Effects of Project on *E. coli* Concentrations

Month	Chippewa Creek at Triangle Island (A3) (cfu/100 mL)			HEPC at Montrose Gate (A2) (cfu/100 mL)			HEPC Above SAB (A5) (cfu/100 mL)			Niagara River Below SAB (A6) (cfu/100 mL)		
	Existing	Future	Difference	Existing	Future	Difference	Existing	Future	Difference	Existing	Future	Difference
January	11.2	11.4	0.18 (1.6%)	306.5	306.4	-0.10 (0.0%)	306.3	306.2	-0.10 (0.0%)	38.2	38.2	0.01 (0.0%)
February	10.2	10.3	0.19 (1.8%)	31.1	31.3	0.16 (0.5%)	31.5	31.6	0.16 (0.5%)	12.0	12.0	0.02 (0.1%)
March	4.3	4.5	0.20 (4.5%)	99.1	99.2	0.10 (0.1%)	99.3	99.4	0.10 (0.1%)	12.5	12.6	0.02 (0.1%)
April	6.4	6.6	0.19 (2.9%)	17.3	17.4	0.17 (1.0%)	17.6	17.8	0.17 (0.9%)	7.2	7.2	0.01 (0.2%)
May	3.3	3.5	0.19 (5.7%)	7.3	7.4	0.18 (2.5%)	7.7	7.8	0.18 (2.3%)	3.6	3.6	0.01 (0.4%)
June	4.1	4.3	0.19 (4.6%)	12.1	12.2	0.17 (1.4%)	12.5	12.6	0.17 (1.4%)	4.6	4.6	0.01 (0.3%)
July	4.0	4.2	0.18 (4.5%)	8.7	8.9	0.17 (2.0%)	9.1	9.3	0.17 (1.9%)	4.4	4.4	0.01 (0.3%)
August	4.5	4.7	0.18 (4.0%)	9.5	9.7	0.17 (1.8%)	9.9	10.1	0.17 (1.7%)	4.9	5.0	0.01 (0.3%)
September	8.8	9.0	0.18 (2.1%)	70.8	70.9	0.12 (0.2%)	71.1	71.2	0.12 (0.2%)	14.0	14.0	0.01 (0.1%)
October	9.9	10.0	0.19 (1.9%)	32.3	32.5	0.16 (0.5%)	32.7	32.8	0.16 (0.5%)	11.6	11.7	0.01 (0.1%)
November	7.1	7.3	0.19 (2.8%)	97.2	97.3	0.10 (0.1%)	97.5	97.6	0.10 (0.1%)	14.4	14.4	0.01 (0.1%)
December	7.8	7.9	0.20 (2.5%)	241.8	241.8	-0.04 (0.0%)	241.7	241.7	-0.04 (0.0%)	26.8	26.8	0.01 (0.1%)
Annual	6.7	6.9	0.19 (2.8%)	77.2	77.3	0.11 (0.1%)	77.4	77.5	0.11 (0.1%)	12.4	12.5	0.01 (0.1%)

4.5 Biochemical Oxygen Demand ($CBOD_5$) and Dissolved Oxygen

The mass balance modelling suggests that the dissolved oxygen concentrations downstream of the discharge are not sensitive to the effluent dissolved oxygen concentrations. As such, effluent dissolved oxygen concentrations equal to 50% of the saturation concentration are recommended as the effluent limit

The recommended annual maximum allowable $CBOD_5$ concentration for effluent is based on the minimum value of 5,876 mg/L (fall) from Table 32. This value is well above the minimum secondary effluent standard compliance limit of 25 mg/L (Table 21). As such, the recommended effluent compliance limit for $CBOD_5$ is 25 mg/L.

Table 32: Maximum Allowable Monthly $CBOD_5$ Concentrations for Discharge to Chippewa Creek

Season	Maximum Allowable Effluent Concentration (mg/L)[1]
January	12,253
February	12,863
March	13,835
April	14,359
May	12,954
June	9,304
July	7,098
August	**5,876**
September	5,883
October	7,030
November	7,960
December	8,969

Notes:

1. Based on effluent dissolved oxygen concentration equal to 50% of saturation.

The predicted effects of the Project on the monthly $CBOD_5$ concentrations in the study are summarized and compared to the predicted worst-case existing conditions in Table 33. The existing conditions are predicted using the mass balance model for low-flow conditions and assume that the existing Niagara Falls WWTP is operating at the rated capacity (68.3 MLD) and discharging effluent with an $CBOD_5$ concentration equal to the monthly 75^{th} percentile of the measured effluent data (5.3 to 11.4 mg/L).

The $CBOD_5$ concentrations are expected to increase by approximately 0.02 mg/L in Chippewa Creek and the HEPC, an increase of 1.2% or less. In the Niagara River, the increases in $CBOD_5$ concentrations are predicted to be less than 0.02 mg/L which represents an increase of 0.1% or less.

Table 33: Summary of Predicted Effects of Project on $CBOD_5$ Concentrations

Month	Chippewa Creek at Triangle Island (A3)			HEPC at Montrose Gate (A2)			HEPC Above SAB (A5)			Niagara River Below SAB (A6)		
	Existing (mg/L)	Future (mg/L)	Difference (mg/L)	Existing (mg/L)	Future (mg/L)	Difference (mg/L)	Existing (mg/L)	Future (mg/L)	Difference (mg/L)	Existing (mg/L)	Future (mg/L)	Difference (mg/L)
January	2.000	2.022	0.022 (1.1%)	2.000	2.022	0.022 (1.1%)	2.008	2.029	0.021 (1.1%)	2.001	2.003	0.002 (0.1%)
February	2.000	2.023	0.023 (1.1%)	2.000	2.022	0.022 (1.1%)	2.009	2.031	0.022 (1.1%)	2.001	2.003	0.002 (0.1%)
March	2.000	2.023	0.023 (1.1%)	2.000	2.022	0.022 (1.1%)	2.010	2.032	0.022 (1.1%)	2.001	2.003	0.002 (0.1%)
April	2.000	2.022	0.022 (1.1%)	2.000	2.021	0.021 (1.1%)	2.008	2.029	0.021 (1.1%)	2.001	2.002	0.002 (0.1%)
May	2.000	2.022	0.022 (1.1%)	2.000	2.021	0.021 (1.1%)	2.010	2.031	0.021 (1.1%)	2.001	2.003	0.002 (0.1%)
June	2.000	2.022	0.022 (1.1%)	2.000	2.021	0.021 (1.1%)	2.010	2.031	0.021 (1.1%)	2.001	2.002	0.002 (0.1%)
July	2.000	2.021	0.021 (1.1%)	2.000	2.020	0.020 (1.0%)	2.008	2.028	0.020 (1.0%)	2.001	2.002	0.002 (0.1%)
August	2.000	2.021	0.022 (1.1%)	2.000	2.021	0.021 (1.0%)	2.017	2.038	0.021 (1.0%)	2.001	2.003	0.002 (0.1%)
September	1.999	2.021	0.022 (1.1%)	1.999	2.020	0.021 (1.1%)	2.019	2.040	0.021 (1.0%)	2.002	2.003	0.002 (0.1%)
October	2.000	2.023	0.022 (1.1%)	2.000	2.022	0.022 (1.1%)	2.016	2.038	0.022 (1.1%)	2.001	2.003	0.002 (0.1%)
November	2.000	2.023	0.023 (1.2%)	2.000	2.022	0.022 (1.1%)	2.009	2.031	0.022 (1.1%)	2.001	2.003	0.002 (0.1%)
December	2.000	2.023	0.023 (1.2%)	2.000	2.022	0.022 (1.1%)	2.007	2.030	0.022 (1.1%)	2.001	2.002	0.002 (0.1%)
Annual	2.000	2.022	0.022 (1.1%)	2.000	2.021	0.021 (1.1%)	2.011	2.032	0.021 (1.1%)	2.001	2.003	0.002 (0.1%)

Notes:

1. SAB – Sir Adam Beck GS

4.6 Total Suspended Solids (TSS)

The annual 75th percentile upstream TSS is 11.3 mg/L suggesting that Chippewa Creek does not typically have high concentration of suspended solids. The mass balance modelling results provided in Table 34 show that, the recommended annual maximum allowable TSS concentration for effluent is the December minimum value of 4,932. This value is well above the minimum secondary effluent standard compliance limit of 25 mg/L. As such, the recommended effluent compliance limit for TSS is 25 mg/L.

Table 34: Maximum Allowable Monthly TSS Concentrations for Discharge to Chippewa Creek

Season	Maximum Allowable Effluent Concentration (mg/L)[1]
January	5,178
February	5,102
March	5,023
April	5,241
May	5,241
June	5,213
July	5,420
August	5,350
September	5,254
October	5,133
November	4,963
December	**4,932**

4.7 Recommended Effluent Objectives

Based on the preceding discussions, a summary of the recommended effluent concentrations for the Chippewa Creek discharge is presented in Table 35.

- While background phosphorus concentrations can exceed PWQO during some months, effluent TP compliance limits for the new plant are recommended based on a well operated secondary treatment facility with phosphorus removal based on the following rationale:
- On an annual basis, there is sufficient capacity to accept an effluent concentration greater than 0.75 mg/L.
- The effluent flow rate represents less than 0.1% of the total flow in Chippewa Creek and as such the contributions of the proposed discharge will cause negligible increase in the total phosphorus concentrations within Chippewa Creek and the HEPC.
- The elevated phosphorus concentration in Chippewa Creek are only experienced during October to February, which is outside the algae growing season. Furthermore, the elevated background phosphorus concentrations are the result of factors outside the study area (e.g., inflow from the Niagara River and Lyons Creek).
- Similarly, the effluent flow rate is insignificant when compared to the flow in the Niagara River below the Sir Adam Beck GS.

Table 35: Summary of Development of Effluent Compliance Limits for Preferred Discharge Location into Chippewa Creek

Parameter		Limiting Assimilative Capacity Concentration[1]	Typical Treatment Plant Effluent[2]	Proposed Effluent Compliance Limits
Total Phosphorus (mg/L)		No capacity[3]	0.5	0.75
Unionized Ammonia (mg/L)		0.1	--	0.10
Total Ammonia (mg/L)	May to October	0.83[4]	<1	8.8
	November to April	3.48[4]	<3	15.0
E. coli (cfu/100 mL)		75,315[5]	200	200
Dissolved Oxygen (% of Saturation)		50%	>80%	N/A[6]
$CBOD_5$ (mg/L)		5,876	25	25
Total Suspended Solids (mg/L)		4,932	25	25

Notes:

1. Lowest seasonal value from local and system compliance points.
2. Typical effluent for a conventional activated sludge without filtration.
3. No capacity – Policy 2 receiver during winter months only.
4. Limits based on acute end-of-pipe toxicity limit of 0.1 mg/L unionized ammonia adjusted for monthly water temperature and pH.
5. Minimum allowable effluent concentration for *E. coli* based on assimilative capacity in Chippewa Creek
6. Not applicable – typical effluent is expected to be better than the limiting assimilative capacity concentration.

5.0 CONCLUSIONS

Based on the analysis in this report, the following conclusions are provided:

- Elevated total phosphorus concentrations in the Niagara River lead to effluent constraints during the winter.
- Degraded water quality in the Welland River East leads to periodic effluent constraints related to total phosphorus and *E. coli* at the system compliance point.
- The recommended effluent objectives and limits for total and unionized ammonia are defined by the end-of-pipe acute toxicity criteria for unionized ammonia (0.1 mg/L) and not by receiving water limitations.
- As expected, summer is the most restrictive season for total ammonia.
- For all other parameters (nitrate, *E. coli*, $CBOD_5$, dissolved oxygen, and TSS) the maximum allowable effluent concentrations at the local and system compliance points are greater than the expected effluent concentrations from a conventional activated sludge treatment plant and so treatment-based limits are recommended.
- The expected water quality concentrations in the receiving waters are not expected to be measurably different from the existing conditions throughout the study area.
- The conceptual outfall design that includes duckbill valves provides reasonable performance for most of the scenarios modelled. The only exception is during high effluent flow rates (120 MLD) during the summer when plume dilution does not reach 200:1 within 1,000 m. However, high effluent flow rates are expected to occur infrequently and have a duration of a few hours or less.
- In periods when the plume has a tendency to sink (June to September), sinking jets produces less dilution factors closer to outfall and the distance to a 200:1 dilution is approximately 350 m
- In periods when the plume has a tendency to float (October to May), a 200:1 dilution is expected within 5 m of the outfall.
- In general variations in the Chippewa Creek flow and effluent flow rate are not expected to noticeably affect the performance of the outfall design.

Table 36 summarizes the proposed effluent objectives and compliance limits for the new 30 ML/d South Niagara Falls WWTP discharging to Chippewa Creek.

Table 36: Recommended Effluent Objectives and Limits for Preferred Discharge Location into Chippewa Creek

Parameter		Effluent Objectives	Effluent Limits
Total Phosphorus (mg/L)		0.5	0.75
Total Ammonia (mg/L)[1]	May to October	6.5	8.8
	November to April	12.0	15.0
E. coli (cfu/100 mL)		200	200
$CBOD_5$ (mg/L)		15	25
Total Suspended Solids (mg/L)		15	25

Notes:

1. Limits based on acute end-of-pipe toxicity limit of 0.1 mg/L unionized ammonia adjusted for monthly water temperature and pH.

6.0 LIMITATIONS

Golder has prepared this report for the exclusive use by the Niagara Region and other members of the Project team for the South Niagara Falls Wastewater Solutions Schedule C Class EA Project. The results presented in this report are for a proposed wastewater treatment plant with a specific design capacity of 30 MLD discharging to the Chippewa Creek location identified Screening Level ACS (Appendix A 2019). The results presented in this report should not be used to assess other design capacities or discharge locations in any way.

Information, analysis, and commentary presented in this report regarding wastewater treatment technologies and the associated typical effluent quality have been provided by CIMA+.

The assessment has been completed using data and information collected and provided by others. Golder does not assume any responsibility related to the accuracy or reliability of the data or information.

Water quality modelling requires the use of many assumptions due to the uncertainty related to determining the physical and chemical characteristics of a complex system. The prediction of water quality is based on several inputs (flows and chemistry), all of which have inherent variability and uncertainty.

GoldSim derives a maximum allowable concentration distribution for each parameter and location by combining randomly sampled flows over numerous (1,000s) of cycles using a Monte Carlo approach. While this approach is valuable because it considers numerous combinations, it may be inaccurate if certain environmental conditions are less represented in historic data than others.

The conventional mass balance ACS approach calculates the maximum allowable effluent concentration for a specific case where the low-flow condition (e.g., 7Q20) occurs for all the inflows at the same time. This is the approach that is typically requested by the MECP and is assumed to represent a worst-case scenario. However, because of the range of the inflow watershed sizes (e.g., Niagara River compared to Lyons Creek), it is highly unlikely that low-flow conditions will occur in all the inflows at the same time.

In natural systems and complex man-made systems, observed conditions will almost certainly vary with respect to estimated conditions. Water quality and flow data has shown a vast range of variability across seasons and locations. This variability may not be captured by the flow and water quality statistics (e.g., 75^{th} percentile concentrations) used as inputs to the models. This is especially true for data sets with small sample sizes.

The mixing zone assessment was completed using a commercially available software package (CORMIX). CORMIX is an expert system that uses the results of a series of laboratory measured plumes (referred to as modules in CORMIX documentation) to represent the release of effluent into a receiving water. Depending on the conditions for individual scenarios (e.g., differences to plume buoyancy), CORMIX can toggle between modules and predict different plume behaviour for these conditions. While CORMIX is regarded as one of the best software packages available for modelling effluent outfall, the results should be interpreted with caution. Golder assumes no responsibility related to the accuracy and reliability of CORMIX.

Since the information regarding the expected effluent quality from various treatment technologies is not site specific, more detailed assessments should be completed prior to the final selection of the required technology.

This assessment is one part of a larger project to select the location and effluent criteria for the proposed WWTP. The results of this assessment should be used in conjunction with the other components of the Project to support any decisions. Given all the inherent uncertainties provided, the results should be used as a tool to aid in the design and planning of the proposed wastewater treatment plant rather than to provide absolute water quality predictions.

Signature Page

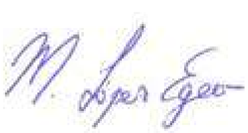
We trust that this report meets your needs at this time. If you have any questions, please do not hesitate to contact the undersigned.

Yours truly,

CIMA+

Troy Briggs, MEng, PEng
Manager, Wastewater

Golder Associates Ltd.



Marta Lopez-Egea, MASc
Water Resources Specialist

Gerard van Arkel, MEng, PEng
Associate, Senior Water Resources Engineer

TB/MLE/GR/GVA/hp/wlm/mp

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APPENDIX A

Screening Level Assimilative Capacity Study

REPORT

South Niagara Falls Wastewater Solutions Schedule C Class Environmental Assessment

Screening Level Assimilative Capacity Study of Discharge Location Alternatives

Submitted to:

Niagara Region

1815 Sir Isaac Brock Way
Thorold ON L2V 4T7

Submitted by:

Golder Associates Ltd.

6925 Century Avenue, Suite #100, Mississauga, Ontario, L5N 7K2, Canada

+1 905 567 4444

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ACRONYMS AND ABBREVIATIONS

Acronym or Abbreviation	Description
ACS	Assimilative Capacity Study
BOD_5	Biochemical Oxygen Demand
$CBOD_5$	Carbonaceous Biochemical Oxygen Demand
CCME	Canadian Council of Ministers of the Environment
CSO	Combined Sewer Overflow
E. coli	*Escherichia coli*
EA	Environmental Assessment
ECA	Environmental Compliance Approval
GS	Generating Station
HEPC	Hydro Electric Power Canal
ICD	International Control Dam
INCW	International Niagara Control Works
MECP	Ministry of the Environment, Conservation and Parks
MOEE	Ministry of Energy and Environment
NOAA	National Oceanic and Atmospheric Administration
NPCA	Niagara Peninsula Conservation Authority
NYPA	New York Power Authority
OPG	Ontario Power Generation
the Project	South Niagara Falls Wastewater Solutions Schedule C Class EA
PWQMN	Provincial Water Quality Monitoring Network
PWQO	Provincial Water Quality Objectives
SLSMC	St. Lawrence Seaway Management Corporation
TSS	Total Suspended Solids
USGS	United States Geological Survey
WSC	Water Survey of Canada
WWTP	Wastewater Treatment Plant

UNITS OF MEASURE

Symbol or Unit	Description
cfs	Cubic feet per second
cfu	Colony-forming unit
kg/d	kilograms per day
km	kilometre
km^2	Square kilometres
m	metre
µg/L	Microgram per litre
mg/L	Milligrams per litre
MLD	Megalitres per day
m^3/s	Cubic metres per second
mL	Millilitre
ºC	Degrees Celsius
%	Percent

1.0 INTRODUCTION

The Regional Municipality of Niagara (Niagara Region) is currently conducting a Schedule “C” Municipal Class Environmental Assessment (EA) for a proposed Wastewater Treatment Plant (WWTP) in the vicinity of Chippewa Creek, Niagara. As well as providing other ancillary services, Golder Associates Ltd. (Golder) has been retained to conduct an Assimilative Capacity Study (ACS) in support of the South Niagara Falls Wastewater Solutions Schedule C Class EA Project (the Project), which is the subject of this technical report.

1.1 Study Background

With significant future regional growth and urban intensification forecast for the area, the 2017 Niagara Region Master Servicing Plan provided a long-term wastewater solutions strategy to improve the existing collection system and add a new, second wastewater treatment facility in South Niagara Falls that can accommodate phased growth, provide wastewater service to currently subserviced areas, reduce pressure on existing wastewater infrastructure, decrease the magnitude and frequency of untreated combined sewer overflows and WWTP bypasses and, in doing so, enhance overall environmental performance.

Wastewater collection within Niagara Falls is currently facilitated through a number of collection systems and pumping stations. These systems convey the wastewater to the existing Niagara Falls WWTP (sometimes referred to as the Stanley Avenue WWTP). Many of the components of the collection system are nearing their design capacity.

The 2017 Master Servicing Plan identified a number of candidate discharge location for a new WWTP in South Niagara Falls that could potentially accept an effluent discharge rate of up to 30 Megalitres per day (30 MLD).

1.1.1 Study Area Overview and Nomenclature

The extent of this study area was identified as the preferred geographical context for siting the new WWTP for the City of Niagara Falls (GMBP, 2019). As depicted on Figure 1, the study area features a number of potential discharge receivers for assimilating the new WWTP discharge, including:

- the Hydro Electric Power Canal (HEPC);
- the eastern portion of the Welland River East;
- Chippewa Creek; and
- The Canadian shoreline of the Niagara River upstream of the International Control Dam (ICD).

The hydrology of the study area has been highly modified and regulated from the natural predevelopment conditions that existed prior to the 1950s. During the 1950s, the HEPC was constructed from the Welland River (upstream of Horseshoe Falls) to the Sir Adam Beck Generating Station (GS) which discharges to Niagara Gorge. As a result, the flow within last 6.5 km of the Welland River was reversed to direct a small portion of Niagara River flows towards the HEPC. The section from the Niagara River to Triangle Island is referred as Chippewa Creek. The amount of flow that is diverted is primarily determined by the following factors:

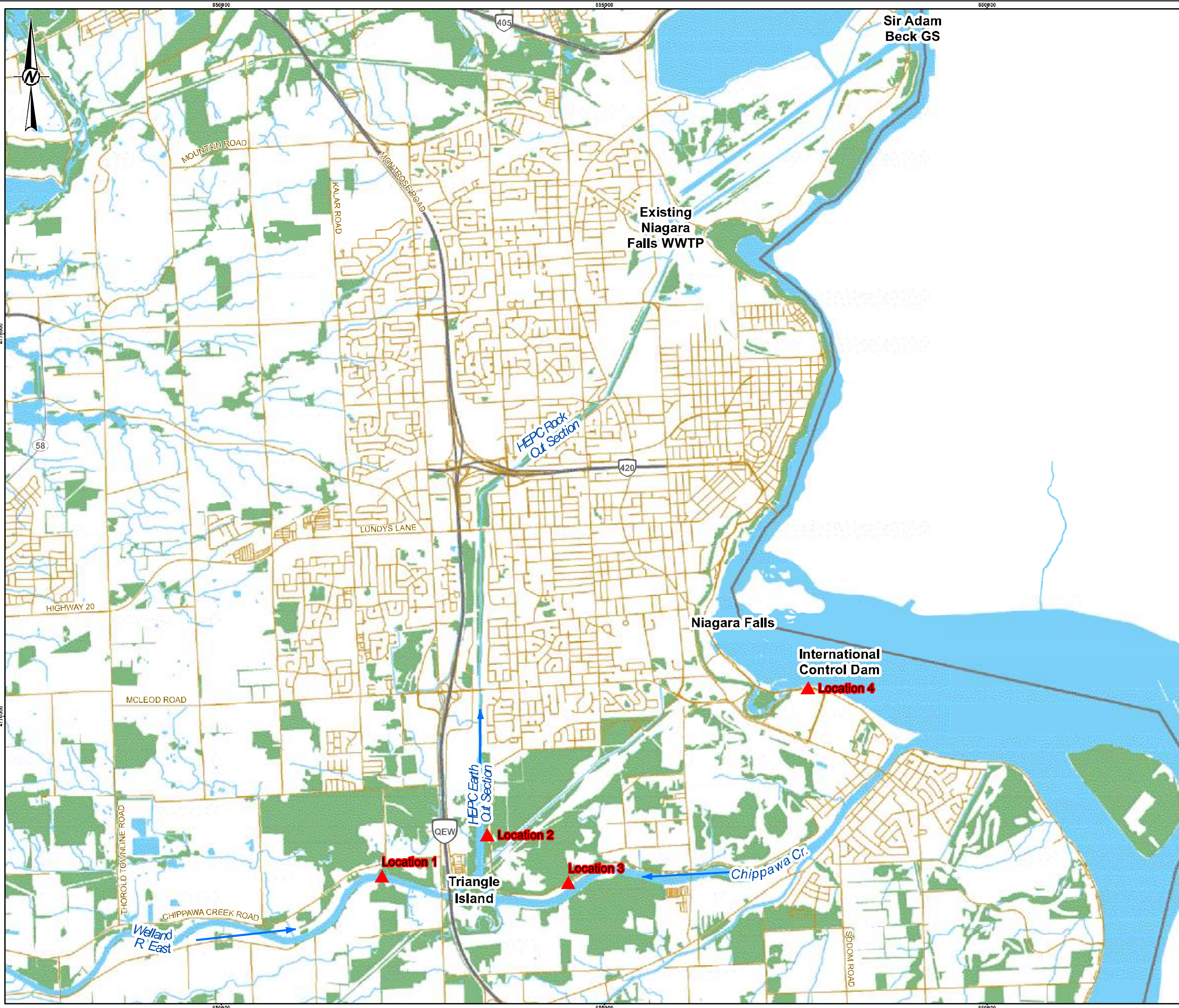
- the operation of the ICD in the Niagara River; which can alternatively increase or decrease the water level in the Niagara River at the mouth of Chippewa Creek; and
- upstream flows in the Niagara River which are determined by water levels at the outlet of Lake Erie, that are influenced by both long-term weather patterns and short-term meteorological events (such as seiching).

The daily operation of the ICD is influenced by the electrical demands and markets in both Ontario and New York State as well as maintaining minimum flow over the falls during tourist periods.

In addition, construction of the Welland Canal to the west of the study area has modified the hydrology and drainage area of the Welland River and several small contributing tributaries. The Welland River passes under the Welland Canal at two locations via siphons that may alter the flow in the river during high flow events. The Lyons Creek watershed area was also decreased by the Welland Canal to the extent that water must now be pumped from the Welland Canal into Lyons Creek to maintain a minimum flow requirement.

For the purposes of maintaining consistent terminology, key surface water features referred to in this ACS use a naming convention adopted by the Ministry of the Environment, Conservation and Parks (MECP), the Niagara Peninsula Conservation Authority (NPCA), and Ontario Power Generation (OPG). Specifically, these key surface water features include:

- **International Control Dam (ICD):** This multi-gated dam in the Niagara River built in 1954 is located approximately 800 m above the Horseshoe Falls and is used to control flows to the Sir Adam Beck GS operated by OPG, the Robert Moses GS operated by the New York Power Authority (NYPA) and the American Falls operated according to Niagara River Treaty (1950). In other literature and documentation, the ICD has sometimes also been referred to as the International Niagara Control Works (INCW).
- **Chippewa–Grass Island Pool (GIP):** This is the area of the Niagara River upstream of the ICD where water levels vary with upstream flow and the operation of the ICD.
- **Hydro Electric Power Canal (HEPC):** This is a canal that conveys diverted flow from the Niagara River (via Chippewa Creek) to the Sir Adam Beck Generating Station.
- **Chippewa Creek:** This is a former portion of the Welland River that flows from the Niagara River to the HEPC when the HEPC is in operation (e.g., reverse flow to natural conditions). During the construction of the HEPC, the width and depth of this section of river were increased to accommodate the increased flow.
- **Triangle Island:** this is a small, constructed island at the junction of the Welland River East, Chippewa Creek, and the HEPC. During normal operation of the HEPC, the diverted flow from the Niagara River flows past the northeast side of Triangle Island from Chippewa Creek into the HEPC while flow from the Welland River East flows past the northwest side of Triangle Island into the HEPC. The channel to the south of Triangle Island is narrower and shallower than the other channels and does not typically have significant flows. Triangle Island is also the location of the safety booms (northeast and northwest sides) used to prevent boat traffic from entering the HEPC.
- **Earth Cut Section:** This is the wide portion of the HECP dug into soil between Triangle Island and the Rock Cut Section of the HEPC and is approximately 1.5 km long.
- **Rock Cut Section:** This is the narrower and deeper section of the HECP cut into bedrock below the Earth Cut Section. The rock cut section of the HEPC is approximately 12 km long and ends at the Sir Adam Beck GS.
- **Welland River East:** This is the portion of the Welland River upstream of triangle island. MECP / NPCA use this convention to distinguish the sections of the Welland River east or west of the Welland Canal.



LEGEND

- APPROXIMATE DISCHARGE LOCATON
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- WATERCOURSE
- INTERNATIONAL BORDER
- WOODED AREA
- WATERBODY

0 1 2

1:50,000 KILOMETRES

NOTE(S)

1. ALL LOCATIONS ARE APPROXIMATE.

REFERENCE(S)

BASE DATA COURTESY OF MNRF LIO. PRODUCED BY GOLDER ASSOCIATES UNDER LICENSE FROM ONTARIO MINISTRY OF NATURAL RESOURCES AND FORESTRY
PROJECTION: UTM ZONE 17N DATUM: NAD 83

CLIENT

REGIONAL MUNICIPALITY OF NIAGARA

PROJECT

SOUTH NIAGARA FALLS WASTEWATER SOLUTIONS
SCHEDULE C CLASS ENVIRONMENTAL ASSESSMENT

TITLE

LOCATION OF PROJECT AREA

CONSULTANT

GOLDER

YYYY-MM-DD	2020-05-19
DESIGNED	MM/PR
PREPARED	PR
REVIEWED	GVA
APPROVED	GVA

PROJECT NO.	CONTROL	REV.	MAP
18104462	0007	A	1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM ANSI B

1.1.2 Potential Discharge Locations

With reference to Figure 1, the ACS considered four different effluent discharge location alternatives for the purpose of receiving treated wastewater effluent discharges from the new WWTP, as follows:

- **Location 1 – Welland River East:** Located immediately west of Triangle Island, the discharge from the new WWTP would mix with flow from Welland River East.
- **Location 2 – Earth Cut Section of HEPC:** Located immediately north of Triangle Island, the discharge from the new WWTP would mix with flow from Chippewa Creek and Welland River East.
- **Location 3 – Chippewa Creek:** Located immediately east of Triangle Island, the discharge from the new WWTP would mix with flow from Chippewa Creek (composed mainly by water from the Niagara River diverted into the HEPC based on flow demand and flow from Lyons Creek) and occasionally with water from Welland River East when the HEPC is not operational.
- **Location 4 – Niagara River:** Located immediately downstream of the ICD and below Chippewa, the WWTP would discharge directly into the Niagara River via a shoreline discharge.

1.2 Study Purpose

The purpose of this ACS is to provide alternatives assessment input in support of the Municipal Class EA by:

1) Evaluating the assimilative capacity of each considered discharge location, considering the seasonal characteristics of key water quality parameters that could be affected by treated effluent discharges at local and system compliance points.
2) Determining the environmental constraints of each discharge location with respect to assimilating a treated wastewater discharge of 30 MLD.
3) Identifying the discharge concentration limits of key water quality parameters to meet Provincial Water Quality Objectives (PWQOs), to meet Canadian Council for Ministers of the Environment criteria (where PWQOs are not available), or to maintain water quality in accordance with MECP Policy 2 requirements conditions at the discharge location.

This study assesses the assimilative capacity and water quality effects at two compliance points for each discharge option. The local compliance point is located immediately downstream of the discharge. In order to consider the cumulative effects of existing discharges to the HEPC, the system compliance point is located in the HEPC immediately downstream of the existing Niagara Falls WWTP and upstream of the confluence with the power tunnels.

1.3 General Study Approach and Report Outline

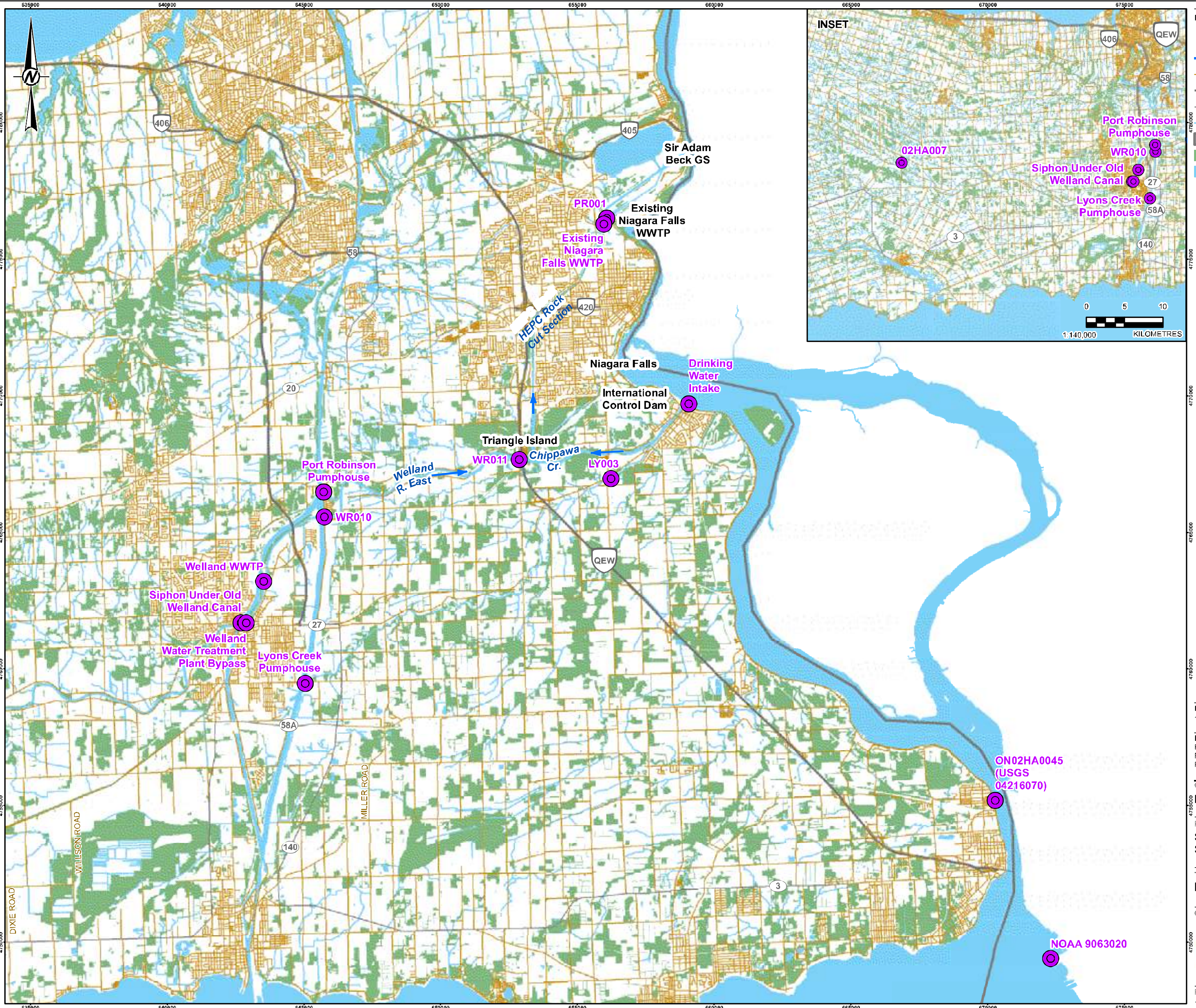
The characterisation of discharge locations considered in this study were based on a number of corporate and publicly available sources including water quality obtained from the MECP Provincial Water Quality Monitoring Network (PWQMN), the US Geological Survey (USGS), The National Oceanic and Atmospheric Administration (NOAA), and the NPCA. Flow data for the Welland River was obtained from the Water Survey of Canada (WSC), flow data for the Niagara River were obtained from the USGS, and flow data for the HEPC were provided by OPG. The structure of this ACS report is presented in the following order:

- Section 2 details the background information obtained and used to characterise seasonal water quality and flow conditions for each of the four discharge locations.

- The hydrological nature of the four locations considered in this study required a slightly modified approach compared to conventional Assimilative Capacity Studies. Namely, system flows at three of the locations (Welland River East, Chippewa Creek and HEPC) are heavily regulated, which meant that the conventional 7Q20 approach to flow derivation was replaced with a stochastic approach. Secondly, the fact that effluent discharges to the Niagara River would only mix with a limited portion of river flow prior to reaching Niagara Falls meant that the mixing potential of effluent discharges at this location were assumed to be limited to only 3% of the Niagara River flows. Section 3 introduces the modelling approach adopted for each discharge location and identifies relevant seasonal and/or environmental constraints, as well as identifying the maximum allowable effluent concentrations at each discharge location to achieve regulatory compliance.
- Based on the constraints identified in Section 3, Section 4 identifies the appropriate treatment technology for each discharge location, presents the ensuing water quality results at each location and provides a high-level discussion of the overall implications on the Project. Section 4 also recommends effluent limits and limits for each location and parameter.
- Section 5 estimates the effects of the Project on the receiving water at selected locations in terms of total phosphorus, nitrate, fecal coliforms (*E. coli*), Carbonaceous Biochemical Oxygen Demand ($CBOD_{5}$), and ammonia (total and unionized).
- Section 6 summarises the key conclusions and recommendations of the ACS.

2.0 BACKGROUND INFORMATION AND DATA REVIEW

This section provides details and summaries of the data used in the ACS. The locations of the monitoring locations where the data were collected are shown in Figure 2.



LEGEND

- REGIONAL SAMPLING STATION LOCATION
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- WATERCOURSE
- INTERNATIONAL BORDER
- WOODED AREA
- WATERBODY

0 5 10
1:140,000 KILOMETRES

NOTE(S)
1. ALL LOCATIONS ARE APPROXIMATE.

REFERENCE(S)
BASE DATA COURTESY OF MNRF LIO. PRODUCED BY GOLDER ASSOCIATES UNDER LICENSE FROM ONTARIO MINISTRY OF NATURAL RESOURCES AND FORESTRY
PROJECTION: UTM ZONE 17N DATUM: NAD 83

CLIENT
REGIONAL MUNICIPALITY OF NIAGARA

PROJECT
SOUTH NIAGARA FALLS WASTEWATER SOLUTIONS
SCHEDULE C CLASS ENVIRONMENTAL ASSESSMENT

TITLE
DATA COLLECTION AND MONITORING LOCATIONS

CONSULTANT
GOLDER

YYYY-MM-DD	2020-05-19
DESIGNED	MM/PR
PREPARED	PR
REVIEWED	GVA
APPROVED	GVA

PROJECT NO.	CONTROL	REV.	MAP
18104462	0007	A	2

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM ANSI B

2.1 Hydrology and Flow Data

2.1.1 Water Management in Study Area

The flow in Chippewa Creek and the HEPC has been controlled since 1921. The ICD has been in operation since 1954 and is jointly funded and controlled by OPG and NYPA in accordance with the 1950 Niagara Treaty (Canada, 1950) and a Memorandum of Understanding between the two power companies which are intended to maximize the beneficial use of the hydro electric potential of the Niagara River, while maintaining the scenic value of Niagara Falls for tourism and other uses of water in the Niagara River. The treaty stipulates that:

- Scenic flow is allocated first, domestic use second, navigational requirements third, and power generation fourth.
- Any river flow diverted for hydro electric power is to split equally between both countries.
- During tourist times, the flow over the falls must be at least 2,832 m^3/s (100,000 cfs). Tourist times are defined as 8 AM to 10 PM from April 1 to September 15 and 8 AM to 8 PM from September 16 to October 31.
- The specified minimum flow over the falls is at least 1,416 m^3/s (50,000 cfs) at all other times.
- If the upstream flow in the Niagara River is less than the specified minimum flows, no river flow is to be diverted to the power canals.

Water levels in the Chippewa-Grass Island Pool are regulated in accordance with the 1993 Directive of the International Niagara Board of Control.

In addition, OPG is required to maintain a minimum flow of 240 m^3/s to the HEPC via Chippewa Creek to ensure that water from the Niagara River reaches the existing drinking water intake of the City of Niagara Falls Water supply plant located near the junction of Chippewa Creek and the Niagara River (Kowalski 2019). Niagara Region is currently in the process of relocating the water supply intake to the Niagara River upstream of Chippewa Creek.

2.1.2 Welland River East

In general, low flow frequency analysis of natural flows is used to generate the low-flow conditions (7Q20) to assess the assimilative capacity of the receiving water body (MOE 1994a). The Welland River East, however, is a complex hydrologic system characterized by natural flows and supplemental flows and the low-flow conditions are dominated by the supplemental flows. As a result, the 7Q20 would not be applicable for this specific assessment. Previous Assimilative Capacity Studies in the Welland River East have successfully applied an approach where the low flows conditions are based on combination of natural and supplemental flows as shown in the ACS completed for the Welland Wastewater Treatment Plant (XCG 2007).

2.1.2.1 Natural Flows in the Welland River East

Regional station data was used to estimate natural flow for the Welland River East. Flow data for the Welland River below Caistor Corners (station 02HA007) from the WSC are available from 1957 to 2017. Flows at the site are calculated based on the prorated watershed area of the site (906 km^2) and the total watershed area of the gauged station (223 km^2). Natural flows in the system are generally low with punctual peak flows recorded during storm events and snowmelt.

Since supplemental flows are significantly higher than average natural flows in the system (i.e., approximately double the annual average flows), natural flows in the Welland River East become relevant only under peak flow conditions. Therefore, flows were prorated between the gauging station (223 km^2) and the area at the site

(906 km^2) according to the Transposition of Flood Discharges Method (MTO, 1997) applying a coefficient of 0.75 to represent peak flows (the coefficient used for average and low flows is 1.0).

The estimated natural flows yield an average annual flow of 6.50 m^3/s with estimated maximum and minimum flows in the range of 132.41 m^3/s and 0.046 m^3/s. The 7Q20 for the natural flows based on the Log Pearson Type III distribution would yield 0.004 m^3/s.

2.1.2.2 Supplemental Flow from Welland Canal into Welland River East

Supplemental flows enter the Welland River East from the Welland Canal (St. Lawrence Seaway Management Corporation [SLSMC] 2019) as follows:

- A series of ports in the roof of the old syphon provide flow from the canal into the river. Depending on the season and water levels in the canal, the total flow ranges from 5 to 7 m^3/s.
- A pump at Port Robinson provides a flow of 0.97 m^3/s to a side channel of the Welland River East, which was cut-off from the main branch of the river during the straightening of the canal in the 1950s.
- The bypass of the Welland Water Treatment Plant provides a flow between the canal and the river that ranges from 4 m^3/s to 6 m^3/s.
- The effluent from the Welland Wastewater Treatment Plant provides a flow of 0.8 m^3/s (XCG 2007).

In general, the supplemental flows from the Welland Canal are from Lake Erie and have better water quality than that of the upstream areas of the Welland River.

Monthly estimates of the supplemental flows for the siphon ports, Port Robinson Pump, the Welland Water Treatment Plant and the Welland WWTP were provided by the SLSMC (SLSMC 2019) for the period 2014 to 2019 and are summarized in Table 1.

Table 1: Summary of Supplemental Flows from Welland Canal into the Welland River East

Source	Winter		Spring		Summer		Fall	
	Min	Avg	Min	Avg	Min	Avg	Min	Avg
Old Welland Canal at Old Siphon[1]	5.17	5.82	5.85	6.61	6.68	6.88	5.56	6.88
Welland Water Treatment Plant[1]	4.45	5.05	4.61	5.65	5.19	5.87	5.64	5.92
Port Robinson Pump[1]	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97
Welland WWTP[2]	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80
Total[3]	**11.39**	**12.64**	**12.23**	**14.03**	**13.64**	**14.52**	**12.97**	**14.57**

Notes:

1. SLSMC 2019.
2. XCG 2007.
3. All flow values in Table 1 are presented in m^3/s.

2.1.3 Niagara River

Daily flow data for Niagara River at Buffalo, New York (opposite Fort Erie, Ontario) were obtained from the USGS for Station 04216000 located in the Niagara River at Buffalo, New York for the years 1926 to 2018 (93 years).

As shown in Table 2, the monthly average flows for the Niagara River at Buffalo range from 5,501 m^3/s (February) to 6,139 m^3/s (May) with an average flow is 5,808 m^3/s. The peak daily flow over the period of record for fall, winter, summer, and spring are 8,466 m^3/s, 9,825 m^3/s, 7,957 m^3/s, and 8,410 m^3/s, respectively. In general, the flows are seasonally consistent year-round with only a slight increase during the spring.

The average daily flow in the Niagara River at Fort Erie did not fall below the tourist time minimum daytime (tourist hours) flow requirements of 2,832 m^3/s (see Section 2.1.1) over the 93-year data period suggesting that there is consistently excess flow available for power generation (e.g., excess flow above treaty requirements).

2.1.3.1 Flow Diversions

Flow diversions from the Niagara River into Chippewa Creek are controlled by OPG based on the requirements in the Treaty for equitable streamflow apportioning between OPG and NYPA. NYPA flows are adjusted upwards to reduce the benefit to OPG at Niagara for the Ogoki-Long Lac diversion south into Great Lakes watershed since mid-1940's

Total diversion flow (HEPC plus three tunnels) data was obtained from OPG for the period 2016 to 2018. As shown in Table 2, the monthly average total flow diversions by OPG range from 1,461 m^3/s (March) to 1,645 m^3/s (August) with an average flow of 1,540 m^3/s. As mentioned previously, the diverted flows by NYPA would be equal to the OPG diverted flows. Instantaneous (hourly) flows ranged from 1,014 m^3/s to 2,272 m^3/s.

Table 2: Average Flow Data for Niagara River at Fort Erie, Diverted Flow by OPG, and Flow Over Niagara Falls

Month	Season	Niagara River at Fort Erie[1] (m^3/s)		Total OPG Diverted Flow[2,3] (m^3/s)		Estimated Flow over Niagara Falls[4] (m^3/s)			
		Monthly Average	Season Average	Monthly Average	Season Average	Monthly Average	Season Average	Monthly Min	Season Min
Jan	Winter	5,573	5,583	1,562	1,521	2,627	2,687	2,124	2,124
Feb		5,501		1,541		2,598		2,124	
Mar		5,667		1,461		2,828		2,124	
Apr	Spring	5,908	6,055	1,493	1,499	2,993	3,101	2,242	2,242
May		6,139		1,479		3,210		2,242	
Jun		6,115		1,526		3,095		2,242	
Jul	Summer	6,023	5,899	1,637	1,619	2,836	2,762	2,242	2,124
Aug		5,909		1,645		2,735		2,242	
Sep		5,760		1,573		2,712		2,124	
Oct	Fall	5,672	5,690	1,464	1,519	2,799	2,738	2,124	2,124
Nov		5,685		1,498		2,763		2,124	
Dec		5,715		1,595		2,654		2,124	
	Annual	**5,808**		**1,540**		**2,822**		**2,124**	

Notes:

1. Measured daily flows for Niagara River at Buffalo, New York (USGS Station 04216000) from 1926 to 2018.
2. Total diverted flow diverted by OPG for 2016 to 2018 (Kowolski, 2019).
3. As per the 1950 Niagara Treaty, diverted flows by NYPA would be equal to the OPG diverted flows.
4. Estimated flow over Niagara Falls based on Niagara River flow, diverted flows by OPG and NYPA, and 1950 Niagara Treaty requirements.

2.1.3.2 Estimated Flow Over Falls

For an evaluation of Location 4, the flow over Niagara Falls (e.g., below the ICD) was based on the following assumptions and methods:

- As per the Niagara Treaty, on any day the flow diverted by NYPA was assumed to be equal to that diverted by OPG.

- While the operation of the ICD may disproportionately affect the flow at Location 4 depending on which gates are closed, it was assumed that the flow downstream of the ICD is distributed equally across the width of the Niagara River.
- Monthly average total diverted flows were estimated based on the data provided by OPG (2016 to 2018).
- The minimum flow requirements of the Niagara Treaty were converted to a time-weighted daily average minimum flow requirement (2,242 m^3/s from April 1st to September 15th and 2,124 m^3/s from September 16th to March 31st).
- Daily average flows over the falls were estimated for the long-term flow record at Buffalo (1926 to 2018) by subtracting the average monthly total diverted flows. If the resulting flow was less than the appropriate daily average minimum flow requirement, then the minimum flow requirement was used (e.g., assumed reduction in diverted flow).

The estimated seasonal and monthly flows over Niagara Falls are also provided in Table 2. The monthly average flows over Niagara Falls range from 2,598 m^3/s (February) to 3,210 m^3/s (May) with an average flow is 2,822 m^3/s.

Restrictions in the total diverted flow by OPG and NYPA occurred approximately 22% of the time between 1926 and 2018 in order to meet the required minimum daily average flow over the falls. These restrictions occurred most frequently during January and February (approximately 33% of the time) and least frequently in May (approximately 8% of the time).

Since the flow over the falls is regulated, a statistical analysis of the flows to determine the 7Q20 low-flow condition is not appropriate. As such, the low-flow condition over the falls was assumed to be the minimum regulated daily average flow over the falls as outlined in the Niagara Treaty (2,242 m^3/s during the tourist season and 2,124 m^3/s during the non-tourist season) that occurs in each assessment season.

2.1.4 Lyons Creek

Historically, the drainage area of Lyons Creek extended into the City of Welland. However, during the construction of the Welland Canal, the watershed was split with the western section draining into the Welland Canal. While the eastern section of Lyons Creek still drains into Chippewa Creek, the drainage area was reduced to approximately 88 km^2. As a result of this reduction in drainage area, the natural flows in Lyons Creek are supplemented by the pumping of water from the Welland Canal at the location where the main channel of Lyons Creek was interrupted by the construction of the canal. From April to November, during the shipping season when the Welland Canal is full, the pumping rate is approximately 0.283 m^3/s (SLSMC 2019). From December to March, when sections of the canal are drained, the flow is reduced to approximately 0.142 m^3/s.

Regional station data was used to estimate the natural flows for the Lyons Creek. Flow data for the Welland River Below Castor Corners (station 02HA007) from the WSC are available from 1957 to 2017. Flows at site are calculated based on the prorated watershed area of the site (88 km^2) and the total watershed area of the gauged station (223 km^2).

2.1.5 Hydro Electric Power Canal (HEPC)

Flow from the Niagara River is diverted to the Sir Adam Beck GS from the Chippewa-Grass Island Pool via three tunnels and the HEPC. Under normal operating conditions, each of these conveyances carries approximately one quarter of the total diverted flow. The flow in the HEPC and tunnels can vary hourly and seasonally due to flow variations in the Niagara River, minimum flow requirements over the falls (see Section 2.4.1), electrical demand, and the market price for electricity.

The flow data provided by OPG (Kowalski 2019) represents the total flow diverted by OPG from the Niagara River to the HEPC and the three tunnels. Typically, the flow in the HEPC represents 27% of the total diverted flow.

Hourly flow data provided by OPG for a three-year period (2016 to 2018) was used as a basis for the following observations regarding the flow in the HEPC:

- The hourly flow rate ranged from 292 m^3/s to 624 m^3/s with an average of 429 m^3/s.
- Flow rates are typically highest during the summer months (446 m^3/s) and lowest in the fall (411 m^3/s).
- Typically, the flows are lowest at 4:00 AM (402 m^3/s) and highest at 6:00 PM (456 m^3/s).

2.1.6 Chippewa Creek

Water from the Niagara River is diverted into Chippewa Creek based on the water levels in the Chippewa-Grass Island Pool. Chippewa Creek extends approximately 6.5 km from the Niagara River to Triangle Island. Lyons Creek drains to the south shore of Chippewa Creek approximately 2km west of the Niagara River.

Given the highly regulated system, flow in Chippewa Creek was estimated in the model based on the flow demand in the HEPC and the estimated flows contributing to the system from the Welland River East and Lyons Creek. The estimated flow (diverted from Niagara River) was calculated in the modelling exercise.

2.1.7 Existing Niagara Falls Wastewater Treatment Plant

The daily volume of the water from the existing Niagara Falls WWTP was provided by Niagara Region for the period 2015 to 2018.

The measured daily flow over the period of record for fall, winter, summer, and spring are 0.55 m^3/s, 0.45 m^3/s, 0.49 m^3/s, and 0.53 m^3/s, respectively. For comparison, the existing Niagara Falls WWTP is rated for an average daily flow of 0.79 m^3/s (68,300 m^3/day), a peak flow rate of 1.58 m^3/s (136,400 m^3/day) during dry weather, and 2.37 m^3/s (205,000 m^3/day) during wet weather (MOE, 2010). These rates are well above the average and peak flows observed for the period 2015 to 2018, meaning that the plant was operating under capacity for the period of record.

The existing Niagara Falls WWTP operates at an average flow of approximately 0.472 m^3/s (40,810 $m^{3/}day$). For the ACS modelling, the effluent flow was maintained at the existing rated capacity of 0.79 m^3/s (68,300 m^3/d). The effluent from the plant to the HEPC and immediately upstream from the system compliance point (upstream of Sir Adam Beck GS).

2.1.8 Combined Sewer Overflows (CSOs) and Wastewater Treatment Plan Bypass

Niagara Region has a total of five Regional CSOs discharging into the HEPC from regional pumping stations. Discharges from the CSOs into the HEPC are primary triggered by storm events. The pumping stations associated with these Regional CSOs are Dorchester Road, Drummond Road, Royal Manor, High Lift and existing Niagara Falls WWTP. The existing Niagara Falls WWTP is further differentiated in terms of water quality as direct overflow (i.e., no treatment) and secondary bypass (i.e., primary treatment).

The City of Niagara Falls has a total of three municipal CSOs discharging to the HPEC from their sanitary and storm sewer collection systems. The locations associated with these municipal CSOs are Sinnicks Avenue, Bellevue Street, and McLeod Road. Volume and frequency of CSOs from the City of Niagara Falls has not been made available and therefore, are excluded from this analysis.

Measured CSO flows were provided by Niagara Region for 2015 through 2018. The measured seasonal frequency and magnitude of overflows from these regional CSOs was analyzed for the period of record. The average seasonal overflow volumes per overflow event (and volume% calculated over average CSO flow discharge over the season) and number of events are summarized on Table 3.

In general, the majority of CSO events occur in spring and summer, coinciding with the largest overflow magnitudes. The secondary bypass from the existing Niagara Falls WWTP yields the largest volume and frequency of CSO flows into the system, followed, by the overflow from the existing Niagara Falls WWTP. These two items yield approximately 94.0% (summer) to 99.6% (fall) of the total CSO flows in the system.

Table 3: Summary of Average Seasonal Flow per Event and Average Number of Events per Season

Season	Dorchester Road	Drummond Road	Royal Manor	High Lift	Existing Niagara Falls WWTP Primary Bypass	Existing Niagara Falls WWTP Secondary Bypass
Average Overflow Volume (m^3/event)						
Winter	720(0.3%)	0(0%)	0(0%)	1,820(0.5%)	7,100(2.5%)	9,200(96.7%)
Spring	4,740(0.5%)	140(0%)	970(0%)	6,810(0.7%)	15,700(2.8%)	17,900(95.9%)
Summer	970(3.9%)	220(0.6%)	0(0%)	3,880(1.5%)	4,300(11.4%)	3,200(82.6%)
Fall	1,360(0.4%)	80(0%)	0(0%)	5,020(0.6%)	8,000(2.3%)	14,500(96.7%)
Annual	1,840(0.2%)	160(0%)	970(0.1%)	4,530(0.2%)	9,500(0.9%)	11,200(98.6%)
Average Number of Overflow Events (events/month)						
Winter	1.75	0	0	1.5	1.75	5.25
Spring	3	1.67	1	2.75	4.75	9
Summer	5.25	3.5	1	0.5	3.5	8
Fall	2	1	0	1	2.25	5.5
Annual	3	1.64	2	1.44	3.06	6.94

Notes:

1. Values in brackets indicate the approximate percentage of the total seasonal volume contributed by each source.

2.2 Water Quality Data

Water quality data for the existing Niagara Falls WWTP and receivers were available for several locations. Most of these locations included parameters suitable to the ACS (e.g., basic chemistry, nutrients, metals, temperature, etc.).

For the initial phases of the ACS, the parameters of concern include total ammonia, unionized ammonia, nitrate, phosphorus, *Escherichia coli* (*E. coli*), dissolved oxygen, Carbonaceous Biochemical Oxygen Demand ($CBOD_5$), and Total Suspended Solids (TSS). The assessment also used pH and water temperature estimate unionized ammonia concentration of the reported water quality data using the equations provided by the MECP (Ministry of Energy and Environment [MOEE], 1994).

The data summaries for the locations in the following sections present the 75th percentile values for all the parameters. These percentiles are used in subsequent analysis as follows:

- The 75th percentile values for total ammonia, nitrate, total phosphorus, *E. coli*, dissolved oxygen, $CBOD_5$, and TSS were used as the background concentrations when estimating the maximum allowable effluent concentrations.
- The 75th percentile values of pH and water temperature were used to estimate the maximum allowable concentration of total ammonia in the effluent based on the estimated maximum allowable effluent concentration for unionized ammonia.
- If more than one water quality monitoring station was available for any given flow source, the maximum reported 75th percentile value was used for conservatism in the modelling exercise.

2.2.1 Applicable Water Quality Guidelines

Applicable PWQOs for the parameters discussed in this memorandum are presented in the Table 4 and are discussed in the following points.

- Since the study area is effectively a river, the PWQO for phosphorus for the avoidance of excessive plant growth in rivers and streams (0.03 mg/L) was used.
- Since there is no PWQO for nitrate, the Canadian Council of Ministers of the Environment (CCME) guideline was selected.
- Seasonal temperature and pH values were used to determine the limits for total ammonia based on the PWQO for unionized ammonia.
- Since the Niagara River, Lyons Creek, and Welland River East are all considered warm water aquatic habitat (NPCA 2011), the dissolved oxygen guideline for warm water fisheries was used.
- The PWQO for fecal coliforms (*E. coli*) is for recreational use (e.g., beaches).
- Since the new WWTP is not expected to release a thermal discharge or alter the pH in the receiving waters, water temperature and pH were excluded from the modelling exercise.
- Since there is no PWQO for total suspended solids, the CCME guideline for clear flow (low flow) was selected.

Table 4: Summary of Applicable Water Quality Objectives

Parameter	PWQO or CCME Guideline
Unionized Ammonia	0.0164 mg/L as N[1]
Total Ammonia	Estimated from unionized ammonia criteria based on ambient water temperature and pH using equations in the Provincial Water Quality Objectives (MOEE 1994)
Nitrate	3 mg/L as N[2]
pH	6.5 to 8.5[1]
E. coli.	100 cfu/100mL[1,3]
Total Phosphorus	0.03 mg/L to avoid excessive plant growth in rivers and streams[1]
Dissolved Oxygen	47% of saturation or 4 mg/L above 20ºC for warm water fisheries[1,5]
Total Suspended Solids	During clear flow (low flow): Maximum average increase of 5 mg/L from background levels for longer term exposures (24 hours to 30 days).[2]
Water Temperature	10ºC above background or 30ºC for thermal discharges[1]

Notes:

1. Provincial Water Quality Objectives (MOEE, 1994).
2. Guideline for freshwater aquatic life in CCME Guidelines (CCME, 2014).
3. PWQO for *E. coli* is for recreational use (e.g., swimming beaches).
4. Since the new WWTP is not expected to release a thermal discharge or alter the pH in the receiving waters, water temperature and pH were excluded from the modelling exercise (explicitly) but used to assess capacity in the system for unionized ammonia.
5. Since the Niagara River, Lyons Creek, and Welland River East are all considered warm water aquatic habitat (NPCA 2011), the dissolved oxygen guideline for warm water fisheries was used.

2.2.2 Welland River East

For the water quality assessment of the Welland River East, data from two monitoring stations were used:

- immediately west (upstream) of Triangle Island at Montrose Road (WR011) with available data from 2011 to 2018; and
- further west (upstream), where the Welland River crosses at the Welland Canal (WR010) with data from 2003 to 2018.

Water quality data for the Welland River East was provided by NPCA. A summary of the seasonal water quality values for WR010 and WR011 are presented in Table 5. Water quality in the Welland River East consistently exceeds the PWQO guidelines for phosphorus and *E. coli*.

As mentioned in Section 0, the flows in the Welland River East are supplemented by flows from the Welland Canal. As a result, the water quality in the Welland River East is a combination of water from the Welland Canal which is effectively water from Lake Erie) and natural drainage from the upper sections of the Welland River Watershed. The water from the canal is typically of better quality than that of the upper Welland River (e.g., lower phosphorus concentrations). The contributions of the Welland Canal flows on the water quality in the Welland River East are demonstrated on Figure 3 when the natural flows are low and diluted by Welland Canal flows, the total phosphorus concentrations are low (e.g., less than 0.05 mg/L). During higher natural flows, the dilution by the canal flows are less pronounced and the total phosphorus concentration are elevated (e.g., up to 0.45 mg/L).

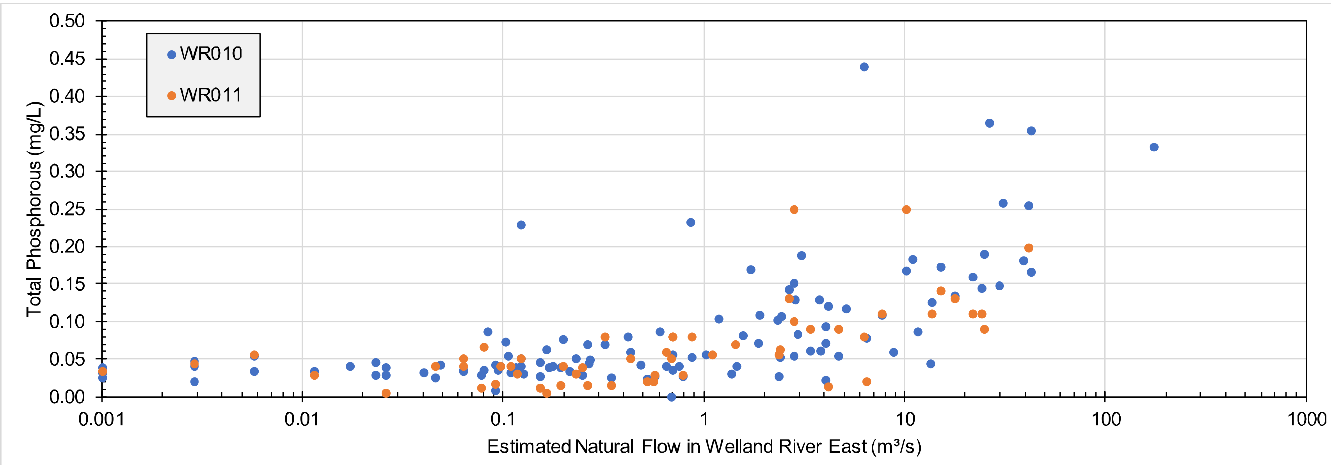


Figure 3: Total Phosphorus Concentration Against Estimated Natural Flow in Welland River East

Comparing the 75th percentile concentrations for both stations showed that ammonia concentrations are higher at WR011 during winter/spring and that overall, the concentration of phosphorus is higher upstream in the Welland River (WR010). The remaining parameters do not show significant differences between upstream (WR010) and downstream (WR011) monitoring stations. Based on the data, there are frequent exceedances of the PWQOs for phosphorus and *E. coli.* in the Welland River East.

The GoldSim model uses the monthly 75th percentile of ammonia, *E. coli*, nitrate, and total phosphorus. For each parameter, the highest 75th percentile value from WR011 and WR010 was selected. The decision to use this approach is based on the uncertainty of WR011 (as it would be influenced by flow from Niagara River) and the additional sources which could affect water quality in the reach between WR010 and WR011. Using the highest value of the two stations yields a conservative approach for prediction of assimilative capacity of the system. The assimilative capacity of the system for ammonia is based on the regulatory limit of unionized ammonia, ammonia in the system (based on 75th percentile), and 75th percentile values of pH and temperature.

The seasonal values selected to characterize the water quality in the Welland River East are presented in Table 5.

Table 5: Summary of Seasonal Water Quality Concentrations for Welland River East

Parameter		Winter		Spring		Summer		Fall	
		WR010	WR011	WR010	WR011	WR010	WR011	WR010	WR011
Number of Samples		5	2	34	17	38	16	41	20
Total Ammonia (mg/L)	Geo-mean	0.21	0.47	0.16	0.16	0.14	0.07	0.10	0.10
	75th	0.23	0.59	0.21	0.28	0.22	0.09	0.20	0.16
Unionized Ammonia (mg/L)	Geo-mean	0.001	0.003	0.003	0.004	0.009	0.003	0.004	0.004
	75th	0.001	0.007	0.006	0.007	0.018	0.004	0.009	0.007
Nitrate (mg/L)	Geo-mean	1.78	2.32	0.76	0.62	0.32	0.33	0.50	0.50
	75th	2.29	2.38	1.11	0.91	0.49	0.48	1.05	0.82
E. coli. (cfu/100 mL)	Geo-mean	-	**2474**	-	66	-	25	-	64
	75th	-	**6920**	-	**308**	-	**105**	-	**170**
Total Phosphorus (mg/L)	Geo-mean	**0.09**	**0.12**	**0.07**	**0.05**	**0.06**	**0.04**	**0.06**	**0.04**
	75th	**0.14**	**0.13**	**0.16**	**0.09**	**0.08**	**0.06**	**0.10**	**0.08**
Dissolved Oxygen (mg/L)	Geo-mean	13.73	14.48	11.64	12.04	9.17	9.78	9.84	9.85
	25th	12.68	13.81	10.66	11.48	8.12	8.66	8.51	8.97
$CBOD_5$ (mg/L)	Geo-mean	-	-	-	0.16	-	0.31	-	0.16
	75th	-	-	-	1.03	-	2.00	-	1.00
Total Suspended Solids (mg/L)	Geo-mean	20.2	26.1	12.6	7.4	8.9	5.6	6.6	4.7
	75th	34.9	28.8	20.9	21.0	11.4	11.8	9.7	6.0
Water Temperature (ºC)	Geo-mean	1.78	1.62	7.54	8.77	22.57	23.64	13.52	13.40
	75th	2.10	1.99	14.39	13.46	24.06	25.27	19.69	20.45
pH	Geo-mean	7.82	7.73	8.08	7.98	8.17	8.08	8.18	8.02
	75th	7.82	7.81	8.23	8.16	8.26	8.23	8.27	8.15

Notes:

1. **Bold** values indicate exceedances of applicable PWQO.
2. Data provided by NPCA.
3. Highlighted values correspond with input to the GoldSim model.

2.2.3 Niagara River

The water quality in the Niagara River was quantified by compiling data from three sources since no one location offered a full complement of data for all required parameters. The data sources were:

- The Niagara River at Fort Erie (ON02HA0045) from 1981 to 1999 (total phosphorus, total ammonia, unionized ammonia, nitrate, and pH).
- The Niagara River at Niagara-on-the-Lake (ON02HA0019) from 1975 to 1999 (total phosphorus only, not used as modelling input).
- The raw water intake data for the Niagara Falls Drinking Water Supply Plant from 2016 to 2018 (*E. coli*).
- Water temperatures in the Niagara River were based on hourly measurements taken at Buffalo, NY (Station 9063020) by NOAA between 2007 and 2018.
- Dissolved oxygen and TSS concentrations were obtained from the USGS for station 04216070 (Niagara River at Fort Erie) for the period 2014 to 2019.

Water quality data for the eastern basin of Lake Erie and the Niagara River at Fort Erie were obtained from the Environment Canada website while the water intake data was provided by Niagara Region. Data from NOAA and the USGS were obtained from their respective websites.

Although older than the Lake Erie data, the Niagara River data was selected since the Lake Erie data was collected sporadically and could not adequately define seasonal variations.

In general, the water quality in the Niagara River meets all of the applicable objectives. The only exception was total phosphorus where the 75th percentile concentration of 0.043 mg/L during winter months exceeds the PWQO (0.03 mg/L). This is a consistent annual pattern that occurs throughout the entire data record, with phosphorus below PWQO during all seasons with the exception of winter. The highest monthly total phosphorus concentrations typically occur in December and January.

Measured data regarding TSS and Carbonaceous Biochemical Oxygen Demand ($CBOD_5$) were not available in sufficient quantity to provide seasonal statistical summaries. However, since the water in the Niagara River is typically clear (NYPA, 2005), it is expected that concentrations of TSS and $CBOD_5$ are low. Sixteen samples collected by the USGS provide annual estimates for the geometric mean and 75th percentile TSS values of 5.2 mg/L and 11.3 mg/L, respectively.

The 75th percentile of seasonal values of different parameters for Niagara River and Lake Erie are presented in Table 6.

This study model uses the seasonal 75th percentile values for the Niagara River station for all parameters except dissolved oxygen. The seasonal 75th percentile values for pH and temperature were used to estimate unionized ammonia concentrations. The seasonal 25th percentile values for dissolved oxygen were used.

Table 6: Summary of Seasonal Water Quality Concentrations for Niagara River

Parameter		Winter		Spring		Summer		Fall	
		Niagara River[2]	Raw Water Intake[2]	Niagara River[2]	Raw Water Intake[2]	Niagara River[2]	Raw Water Intake[2]	Niagara River[2]	Raw Water Intake[2]
Number of Samples		596	39	361	39	346	39	375	39
Total Ammonia (mg/L)	Geo-mean	0.007	-	0.029	-	0.022	-	0.012	-
	75th	0.014	-	0.046	-	0.044	-	0.032	-
Unionized Ammonia (mg/L)	Geo-mean	<0.001	-	<0.001	-	0.001	-	<0.001	-
	75th	<0.001		0.001	-	0.002	-	<0.001	-
Nitrate (mg/L)	Geo-mean	0.25	0.20	0.26	0.19	0.19	0.10	0.14	0.07
	75th	0.31	0.36	0.31	0.30	0.26	0.20	0.18	0.12
E. coli. (cfu/100 mL)	Geo-mean	-	5	-	3	-	3	-	5
	75th	-	50	-	12	-	8	-	26
Total Phosphorus (mg/L)	Geo-mean	0.027	-	0.019	-	0.015	-	0.019	-
	75th	**0.043**	-	0.026	-	0.022	-	0.027	-
Dissolved Oxygen[3] (mg/L)	Geo-mean	11.1	-	9.81	-	10.5	-	10.4	-
	25th	10.4	-	8.60	-	8.98	-	8.75	-
Water Temperature (ºC)[4]	Geo-mean	1.5	-	6.4	-	21.7	-	13.8	-
	75th	2.5	-	10.1	-	23.9	-	20.1	-
pH	Geo-mean	7.98	-	8.12	-	8.27	-	8.08	-
	75th	8.12	-	8.20	-	8.33	-	8.20	-

Notes:

1. **Bold** values indicate exceedances of applicable PWQO.
2. Data provided by Niagara Region.
3. Dissolved oxygen data obtained from USGS.
4. Data downloaded from NOAA (NOAA, 2019).
5. Average value – geometric mean could not be calculated due to water temperatures below zero.
6. Shaded cells correspond with input to the GoldSim and Mass Balance models.

The total phosphorus concentrations in the upper section of the Niagara River (Fort Erie) are compared to those on the lower section (Niagara-on-the-Lake) in Table 7 for the period 1981 to 1999. Apart from summer, the mean total phosphorus concentrations in the lower sections are lower than the concentrations in the upper section. In all seasons except winter, the difference in mean and 75th percentile concentrations are less than 0.03 mg/L (3 µg/L) suggesting that the effects of current direct phosphorus loads to the Niagara River (e.g., not from Lake Erie) are not measurable.

Table 7: Comparison of Total Phosphorus in Niagara River Between Fort Erie and Niagara-on-the-Lake

Statistic	Location	Winter	Spring	Summer	Fall
Number of Samples	Fort Erie[1]	597	626	605	618
	Niagara-on-the-Lake[2]	819	865	846	839
Geometric Mean (mg/L)	Fort Erie	0.0346	0.0238	0.0196	0.0241
	Niagara-on-the-Lake	0.0249	0.0206	0.0200	0.0228
75th Percentile (mg/L)	Fort Erie	0.0427	0.0259	0.0215	0.0265
	Niagara-on-the-Lake	0.0345	0.0264	0.0204	0.0257

Notes:

1. Data for Fort Erie collected at Station ON02HA0045 (1981 to 1999).
2. Data for Niagara-on-the-Lake collected at Station ON02HA0019 (1981 to 1999).

2.2.4 Lyons Creek

A summary of measured water quality in Lyons Creek is provided in Table 8. Data were provided by NPCA for station LY003 between 2003 and 2018. $CBOD_5$ data was available only for the 2009 to 2014 period, while dissolved oxygen was not available in the dataset provided for this study.

As expected for a small watershed that drains agricultural areas, the total phosphorus concentrations in Lyons Creek are elevated well above the PWQO.

Table 8: Summary of Seasonal Water Quality Concentrations for Lyons Creek

Parameter		Winter	Spring	Summer	Fall
	Number of Samples	3	35	44	44
Total Ammonia (mg/L)	Geo-mean	0.059	0.051	0.041	0.035
	75th	0.059	0.120	0.080	0.060
Unionized Ammonia (mg/L)	Geo-mean	-	0.002	0.002	0.004
	75th	-	0.005	0.004	0.008
Nitrate (mg/L)	Geo-mean	0.75	0.08	0.07	0.10
	75th	0.87	0.20	0.20	0.20
E. coli. (counts/100 mL)	Geo-mean	**137**	45	32	44
	75th	**520**	95	57	88
Total Phosphorus (mg/L)	Geo-mean	0.147	0.124	0.141	0.103
	75th	**0.255**	**0.160**	**0.160**	**0.140**
$CBOD_5$ (mg/L)	Geo-mean	-	1.16	0.95	1.13
	75th	-	2.00	1.00	1.00
Water Temperature (ºC)	Geo-mean	0.30	6.4	15.1	18.4
	75th	0.30	14.9	26.1	24.7
pH	Geo-mean	7.43	7.83	7.87	7.78
	75th	7.65	7.99	8.02	7.95

Note:

1. **Bold** values indicate exceedances of applicable PWQO.
2. Data provided by NPCA.
3. Shaded correspond with input to the GoldSim and Mass Balance models.

2.2.5 Hydro Electric Power Canal (HEPC)

A summary of the measured water quality in the HEPC near the existing Niagara Falls WWTP is provided in Table 9. Data were provided by NPCA for station PR001 (HEPC at Whirlpool Road) between 2012 and 2018. Based on these data, there are exceedances of the PWQOs for phosphorus during fall and winter months and *E. coli.* in the HEPC.

The GoldSim model does not use this data as input, but these measurements are used to validate the model performance downstream of the existing Niagara Falls WWTP.

Table 9: Summary of Seasonal Water Quality Concentrations in the Hydro Electric Power Canal

Parameter		Winter	Spring	Summer	Fall
	Number of Samples	3	17	17	15
Total Ammonia (mg/L)	Geo-mean	0.078	0.264	0.186	0.209
	75^{th}	0.180	0.375	0.250	0.280
Unionized Ammonia (mg/L)	Geo-mean	0.001	0.004	0.008	0.008
	75^{th}	0.001	0.006	0.015	0.012
Nitrate (mg/L)	Geo-mean	0.37	0.21	0.14	0.12
	75^{th}	0.51	0.27	0.22	0.16
E. coli. (cfu/100 mL)	Geo-mean	**5,780**	**283**	**116**	**570**
	75^{th}	**7,550**	**440**	**220**	**4,200**
Total Phosphorus (mg/L)	Geo-mean	**0.042**	0.013	0.015	0.022
	75^{th}	**0.059**	0.018	0.020	**0.040**
Dissolved Oxygen (mg/L)	Geo-mean	16.37	12.46	10.00	9.07
	25^{th}	13.56	9.88	8.26	6.62
$CBOD_5$ (mg/L)	Geo-mean	-	0.24	0.07	0.57
	75^{th}	-	2.00	0.05	2.00
Total Suspended Solids (mg/L)	Geo-mean	15.4	2.6	2.5	4.7
	75^{th}	19.5	2.8	2.2	14.8
Water Temperature (ºC)	Geo-mean	2.1	11.5	22.4	9.8
	75^{th}	3.5	18.6	23.6	13.5
pH	Geo-mean	7.86	8.00	8.12	8.03
	75^{th}	7.99	8.16	8.22	8.14

Note:

1. **Bold** values indicate exceedances of applicable PWQO.
2. Data provided by NPCA.

2.2.6 Existing Niagara Falls Wastewater Treatment Plant, Primary Bypass, and Secondary Bypass

Water quality data and laboratory analysis were provided for the existing Niagara Falls WWTP Final Effluent from 2015 to 2018 by the Niagara Region. Water quality data for the Plant Bypass (Sewage receives no treatment prior to release) and the Secondary Bypass (Sewage receives primary treatment prior to release) were also provided. The water quality data are summarized in Table 10.

For validation, the GoldSim model uses the largest between the geometric mean and the 75th percentile value to characterize the effluent to the existing Niagara Falls WWTP and the primary and secondary bypass data. The effects of CSOs were included and the water quality was assumed to correspond to values reported for the Plant Bypass. The assimilative capacity of the system was estimated by excluding all CSOs, and assuming that the water quality from the effluent at existing Niagara Falls WWTP correspond with the regulatory limits outlined in the Amended Environmental Compliance Approval (ECA) number 7962-7ZLKR6, issued on February 3, 2010. The regulated parameters which are outlined in the aforementioned ECA are total phosphorus and *E. coli*, with effluent limits specified as at 0.75 mg/L and 200 counts/100 ml, respectively.

The data presented in Table 10 indicates that the 75th percentile of total phosphorus during summer would be exceeding the regulatory requirement outlined in the ECA.

Table 10: Summary of Seasonal Water Quality Concentrations for the Existing Niagara Falls Wastewater Treatment Plant Effluent, Primary Bypass, and Secondary Bypass

Parameter		Winter			Spring			Summer			Fall		
		Effluent	Primary Bypass	Secondary Bypass	Effluent	Primary Bypass	Secondary Bypass	Effluent	Primary Bypass	Secondary Bypass	Effluent	Primary Bypass	Secondary Bypass
Number of Samples		361	7	18	368	18	34	368	14	31	364	9	20
Total Ammonia (mg/L)	Geo-mean	4.04	17.09	18.79	2.91	10.20	15.87	3.66	10.45	20.17	3.69	5.66	14.59
	75th	9.61	33.28	22.83	7.37	19.60	23.50	8.42	19.78	27.80	8.01	18.35	19.65
Unionized Ammonia (mg/L)	Geo-mean	**0.014**	-	-	**0.013**	-	-	**0.026**	-	-	**0.021**	-	-
	75th	**0.032**	-	-	**0.032**	-	-	**0.058**	-	-	**0.046**	-	-
Nitrate (mg/L)	Geo-mean	**6.53**	0.46	0.22	**5.91**	0.44	0.32	**5.38**	0.24	0.22	**5.71**	0.29	0.24
	75th	**9.64**	2.03	0.20	**8.61**	1.70	0.21	**7.65**	0.20	0.21	**7.82**	0.47	0.20
E. coli. (cfu/100 mL)	Geo-mean	7	-	**4,102,000**	9	**1,395,500**	**1,972,600**	6	**4,177,700**	**4,447,900**	8	**2,800,600**	**5,047,200**
	75th	13	-	-	13	**2,550,000**	**3,650,000**	10	**5,802,500**	**8,160,000**	11	**6,995,000**	**8,422,500**
Total Phosphorus (mg/L)	Geo-mean	**0.30**	**3.60**	**5.12**	**0.28**	**2.26**	**3.05**	**0.40**	**3.21**	**3.50**	**0.35**	**2.53**	**3.39**
	75th	**0.38**	**5.87**	**8.08**	**0.36**	**2.98**	**5.18**	**0.52**	**4.35**	**4.40**	**0.47**	**4.60**	**4.53**
$CBOD_5$ (mg/L)	Geo-mean	4.39	68.12	175.41	4.72	71.21	100.42	5.23	105.87	128.56	5.61	90.31	126.15
	75th	5.80	142.75	279.75	6.50	122.50	143.00	7.73	136.25	177.00	8.40	167.00	166.25
Water Temperature (ºC)	Geo-mean	10	-	-	11.9	-	-	20.2	-	-	17.3	-	-
	75th	11.7	-	-	14.5	-	-	21.9	-	-	20.2	-	-
pH	Geo-mean	7.25	-	-	7.29	-	-	7.25	-	-	7.24	-	-
	75th	7.35	-	-	7.4	-	-	7.36	-	-	7.31	-	-

Note:

1. **Bold** values indicate exceedances of applicable PWQO.
2. Data provided by Niagara Region.
3. Shaded cells correspond with input to the GoldSim for verification only

2.3 Total Phosphorus Loads in Study Area

The existing total phosphorus loads in the study area provided in Table 11 were estimated based on seasonal average flows and geometric mean concentrations for background. The estimates show that:

- Over 98% of the total phosphorus in the Niagara River comes from Lake Erie.
- The contributions from the Welland River East represent about 1% of the total phosphorus loads.
- Based on the rated capacity and effluent discharge limits, the existing Niagara Falls WWTP contributes approximately 19 tonnes/year (0.3% of the total).
- Total annual contributions from the primary secondary bypasses at the existing Niagara Falls WWTP and the CSOs are estimated to be less than 2 tonnes/year (less than 0.05% of the total loads in the Niagara River).

Table 11: Estimated Seasonal and Annual Total Phosphorus Loads in Study Area

Season	Winter (kg/d)	Spring (kg/d)	Summer (kg/d)	Fall (kg/d)	Annual (tonnes/year)
Niagara River at Fort Erie	15,066.2	11,036.0	8,952.9	10,748.6	4,173.1 (98.3%)
Niagara River into Chippewa Creek	960.4	622.2	554.9	654.1	254.3 (6.0%)
Lyons Creek	35.1	40.0	10.6	16.8	9.3 (0.2%)
Welland River East	114.7	173.0	88.0	106.3	44.0 (1.0%)
Existing Niagara Falls WWTP Effluent[2]	51.2	51.2	51.2	51.2	18.7 (0.3%)
Existing Niagara Falls WWTP Primary Bypass	0.5	1.6	0.5	0.5	0.3 (0.01%)
Existing Niagara Falls WWTP Secondary Bypass	3.0	4.8	1.3	1.9	1.0 (0.02%)
Combined Sewer Overflows	0.3	1.2	0.3	0.5	0.2 (<0.01%)
HEPC at Sir Adam Beck	1,165.2	893.9	706.9	831.3	327.8 (7.7%)
Total[3]	15,271.0	11,307.7	9,104.8	10,925.8	4,246.6 (100%)

Note:

1. Values in brackets represent percentage of total annual loads to Niagara River not including other inflows.
2. Based on ECA effluent limits (0.75 mg/L) and rated capacity of plant (68.3 MLD).
3. Total does not include contributions from other sources (e.g., other tributaries, discharges to Niagara River, etc.)

2.4 Data Conclusions and Generalizations

Based on the preceding characterisation of available flow and water quality data, the following conclusions are provided:

- There are no major seasonal variations in Niagara River flow. Variations in Niagara River flow are likely related to changes in the water level in Lake Erie. These variations can either be long-term due to seasonal or interannual changes in the regional hydrology and precipitation (e.g., over entire Great Lakes basin) or short-term due to wind related events (e.g. longitudinal seiching) along Lake Erie.
- Flows in the HEPC and Chippewa Creek are controlled by the operation of the ICD and should not be represented as a natural flow regime in the ACS.
- The background concentrations of two parameters, phosphorus and *E. coli*, are shown to exceed their respective water quality criteria within two or more watercourses discharging to the HEPC:
- While the Niagara River generally has lower concentrations of phosphorus when compared to the Welland River and Lyons Creek, it represents a far more significant loading source of this parameter due to the considerable difference in flows directed through the HEPC from all sources:
  - Niagara River approximates 95.1% of background HEPC flows;
  - Welland River (natural and supplemental flows) approximates 4.5% of background HEPC flows;
  - Lyons Creek contributes less than 0.3% of background HEPC flows; and
  - Existing Niagara Falls WWTP approximates 0.1% of background HEPC flows.
- Total phosphorus concentrations within the Niagara River tend to increase substantially outside the growing season; the winter 75th percentile phosphorus concentration in the Niagara River is almost twice that of other seasons (22 to 27 µg/L).
- A comparison of the total phosphorus concentrations in the upper and lower sections of the Niagara River suggest that the current direct phosphorus loads to the Niagara River (e.g., not from Lake Erie) are not measurable.
- Notably, it has recently been estimated that 57% of all phosphorus loads to Lake Ontario come from the Niagara River from upstream sources in Lake Erie (ECCC & USEPA, 2018).
- The Welland River East and Lyons Creek also have some local influence, particularly in spring when background phosphorus loading to the HEPC from these two watercourses alone can exceed 20%.
- Water quality in Welland River East, particularly total phosphorus, deteriorates as the natural flows increase. This correlation is likely attributed to the increased influence of poor land management practices during rainfall runoff compared to the beneficial dilution effects of consistent, supplemental inflows from the Welland Canal via the Port Robinson Pumping Station, ports in the old siphon, and the Welland WWTP bypass under low flow conditions.
- Relative to the Niagara River, bacteriological concentrations in the Welland River and Lyons Creek are so high that the Welland River and Lyons Creek are the dominant sources of *E. coli* throughout the winter and spring, despite order of magnitude differences in flow volume.
- As such, much of the water quality issues in the system are currently being influenced by background contributions from Lake Erie and smaller watersheds located upstream of the HEPC.

3.0 MODELLING APPROACH AND RESULTS

The modelling approach was designed with the following objectives:

- Estimate the remaining capacity of the receiving waters to accept the proposed WWTP effluent flows without exceeding applicable guidelines,
- Estimate the recommended effluent limits for each of the discharge locations and compare those limits to feasible limits based on the available treatment technology, and
- Estimate the existing and future concentrations in the receiving waters at selected locations based on the recommended effluent limit.

Given the complexity of the hydrodynamic conditions in the study area, the first three discharge locations (Location 1 – Welland River East, Location 2 – HEPC and Location 3 – Chippewa Creek) will be modelled using a stochastic approach. The fourth location, evaluating a discharge to the Niagara River, is relatively simple by comparison and was modelled using a mass balance approach.

The following points outline the methods used to complete the ACS at the four locations and for various parameters:

- Given the complex and regulated hydrodynamic conditions in Location 1 – Welland River East, Location 2 – HECP and Location 3 – Chippewa Creek, a stochastic model (GoldSim) was used to complete the ACS for total phosphorus, total ammonia, nitrate, and fecal coliforms (*E. coli*). Estimates for unionized ammonia were calculated based on modelled ammonia and measured 75th percentile temperature and pH.
- To provide an alternate estimate of the assimilative capacity, a mass balance model was developed to estimate the maximum allowable effluent concentrations for total ammonia, unionized ammonia, nitrate, fecal coliforms (*E. coli*), and total phosphorus for conditions where all the flows in the study area were assumed to be representative of low-flow conditions (e.g., 7Q20 or minimum regulated flow).
- The assimilative capacity was assessed at two compliance points; a local compliance point that is immediately downstream of the proposed discharge and a system compliance point in the HEPC downstream of the existing Niagara Falls WWTP to consider cumulative effects in the study area.
- For Location 4 – Niagara River, the effluent is not expected to mix with the entire width if the Niagara River before reaching Niagara Falls. As such a 2-Dimensional Gaussian Plume model was used to predict the lateral mixing of the proposed effluent in the Niagara River. This model was used to assess for total phosphorus, total ammonia, unionized ammonia, nitrate, and fecal coliforms (*E. coli*).
- For parameters associated with oxygen in the water (dissolved oxygen and $CBOD_5$), the maximum allowable effluent concentrations were estimated using a simplified and conservative dissolved oxygen mass balance model that included $CBOD_5$ decay for all the locations. Since a high rate of reaeration is expected in the Niagara River and HEPC due to current speeds, this assessment was only completed for a local compliance point.
- The assimilative capacity did not consider the depletion of dissolved oxygen associated with the nitrification of ammonia.
- A simple mass balance model was used to estimate the maximum allowable effluent concentrations for TSS based on the CCME recommended maximum increase of 5 mg/L over the background conditions (Table 4).

3.1 GoldSim Modelling for Locations 1 Through 3

A stochastic water balance and water quality model was developed using GoldSim version 12.1. GoldSim is a graphical, object-oriented mathematical model where all input flows, constituents and functions are defined by the user and are built as individual objects or elements linked together by mathematical expressions. The object-based nature of the model is designed to facilitate understanding of the various factors, which control an engineered or natural system and predict the future performance of the system.

In GoldSim, each flow that could influence water quality predictions for the Project was itemized and assigned a source term chemistry, for the constituents of interest, based on measured water quality in the system. The model was developed to allow the user to run specific scenarios, including baseline or future conditions (by specifying the desired location of the new WWTP).

3.1.1 Model Conceptualization

The water balance and water quality model were designed to estimate the assimilative capacity and future concentrations in the system. GoldSim runs calculations on a daily timestep for the season of interest.

In GoldSim, each flow (e.g., river flows, discharges, etc.) entering the area of interest and with potential to affect water quantity and/or quality of the system was itemized and assigned a source term chemical profile for selected constituents, based on measured water quality data. Inflow volumes and concentrations were included as inputs to the system to account for loadings from major watersheds, CSOs, and WWTPs draining into the study area.

The stochastic approach was selected to account for the variability and/or uncertainty of the input parameters controlling the model associated with flow. Stochastic modelling in GoldSim was achieved using a Monte Carlo simulation approach. This approach consists of running the model for a selected number of iterations (i.e., realizations). For each realization, the stochastic inputs are randomly sampled based on their statistical distributions. It was assumed that 1,000 realizations would be sufficient to reach a representative and convergent distribution of results. The probability distribution assumed a log-normal distribution for the flows, defined seasonally. By running the model stochastically, each flow will present a range rather than a single value, which accounts for the observed variability in the available dataset.

For the purpose of analysing the flows on a seasonal manner, the months were grouped as follows: March to May to represent spring, June to August to represent summer, September to November to represent fall; and December to February to represent winter. For the purpose of analysing the flows on a seasonal manner, the months were grouped as follows: March 1st to May 31st to represent spring, June 1st to August 31st to represent summer, September 1st to November 30th to represent fall; and December 1st to February 28th to represent winter. While the seasonal patterns varied between flows assessed, the seasonal definition remained unchanged between flow inputs. Average, standard deviation, maximum and minimum flows were used to characterize flow distribution. Flows which did not show seasonal variability were input as a constant value throughout the year.

Water quality concentrations for inflows were based on the 75th percentile seasonal concentrations from measured water quality data for total phosphorus, nitrate, and total ammonia.

Following the model run, the probability of exceedance was calculated based on the 1,000 values calculated at each timestep to assess the range of conditions that could occur in the local and system compliance point for each scenario and season. In a typical ACS, the recommended effluent limits are estimated for a low flow condition that occurs for one week every 20 years (i.e., 7Q20). GoldSim was used to estimate the allowable effluent limits that will result in exceedances of the criteria no more than 5.0% of the time.

Recommended effluent limits were estimated by iteratively running the model to identify a mass flow that results in the water quality in the HEPC meeting PWQO criterion for each of the water quality parameters at the discharge location of the HEPC into the Niagara River. Allowable mass was then converted to the allowable concentration according to the flow in the new WWTP.

3.1.2 Flow Implementation

Flow was implemented in the model based on the available data and the stochastic modelling using the GoldSim model for Welland River East, Lyons Creek, and the HEPC. Flow in Chippewa Creek was estimated using the HEPC flow as well as the flows coming from the Welland River East and Lyons Creek (Sections 2.1.4 and 2.1.5).

3.1.2.1 Welland River East

Table 12 shows the parameters associated with the log-normal distributions followed to characterize the seasonal flow in Welland River East in GoldSim. These distributions include all supplemental inflows from the Welland Canal into the Welland River East. Figure 4 shows the probability distribution of seasonal flows.

Table 12: Summary of Seasonal Flow Statistics for Welland River East Including Supplemental Flows

Parameter	Winter	Spring	Summer	Fall
Mean flow (m^3/s)	17.7	24.4	14.9	20.6
Maximum flow (m^3/s)	29.0	40.2	19.9	32.8
Minimum flow (m^3/s)	14.0	14.7	13.6	12.7
Standard deviation (m^3/s)	3.8	5.4	1.3	4.8

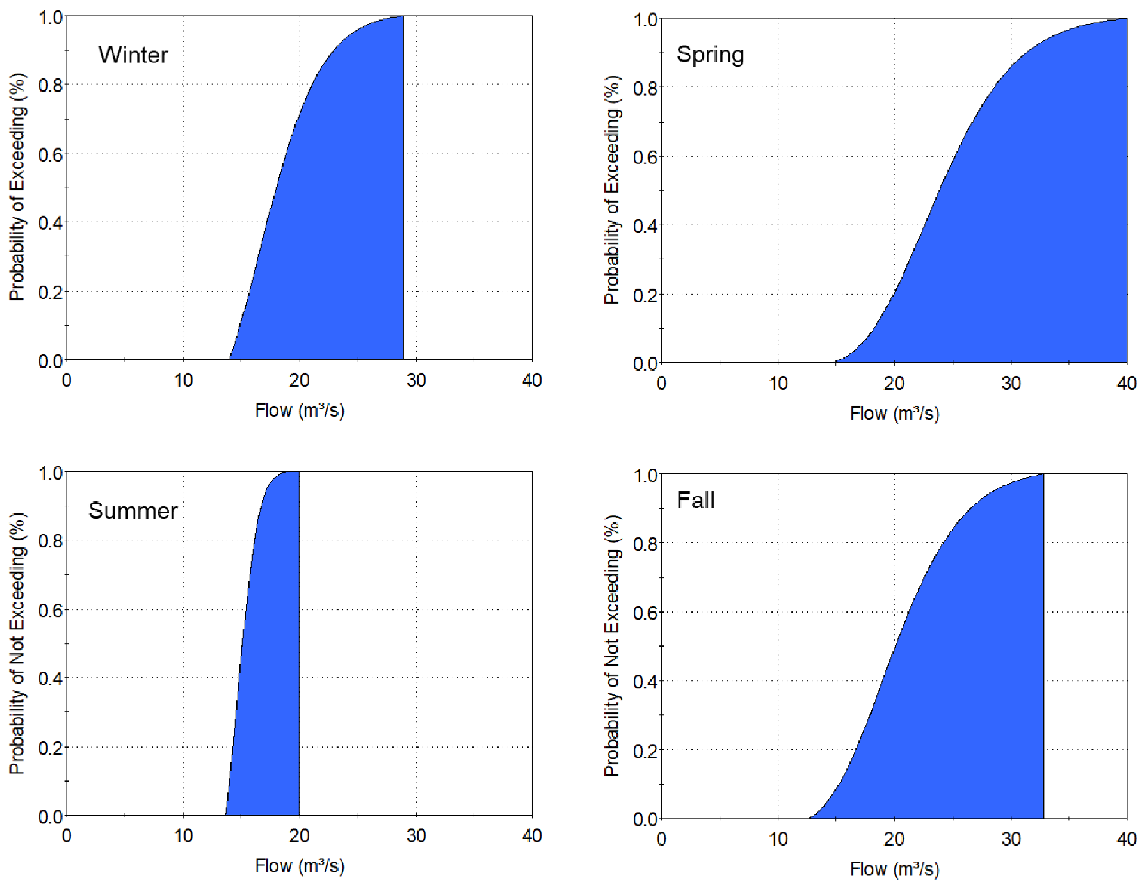


Figure 4: Seasonal Log-Normal Distribution of Flows in the Welland River East Including Supplemental Inflows

3.1.2.2 Lyons Creek

Table 13 shows the parameters associated with the seasonal log-normal distributions followed to characterize the flow in Lyons Creek in GoldSim. Figure 5 shows the probability distribution of seasonal flow.

Table 13: Summary of Seasonal Flow Statistics for Lyons Creek

Parameter	Winter	Spring	Summer	Fall
Mean flow (m^3/s)	1.4	2.0	0.5	0.7
Maximum flow (m^3/s)	3.1	4.0	1.2	2.2
Minimum flow (m^3/s)	0.2	0.6	0.3	0.3
Standard deviation (m^3/s)	0.7	0.7	0.2	0.5

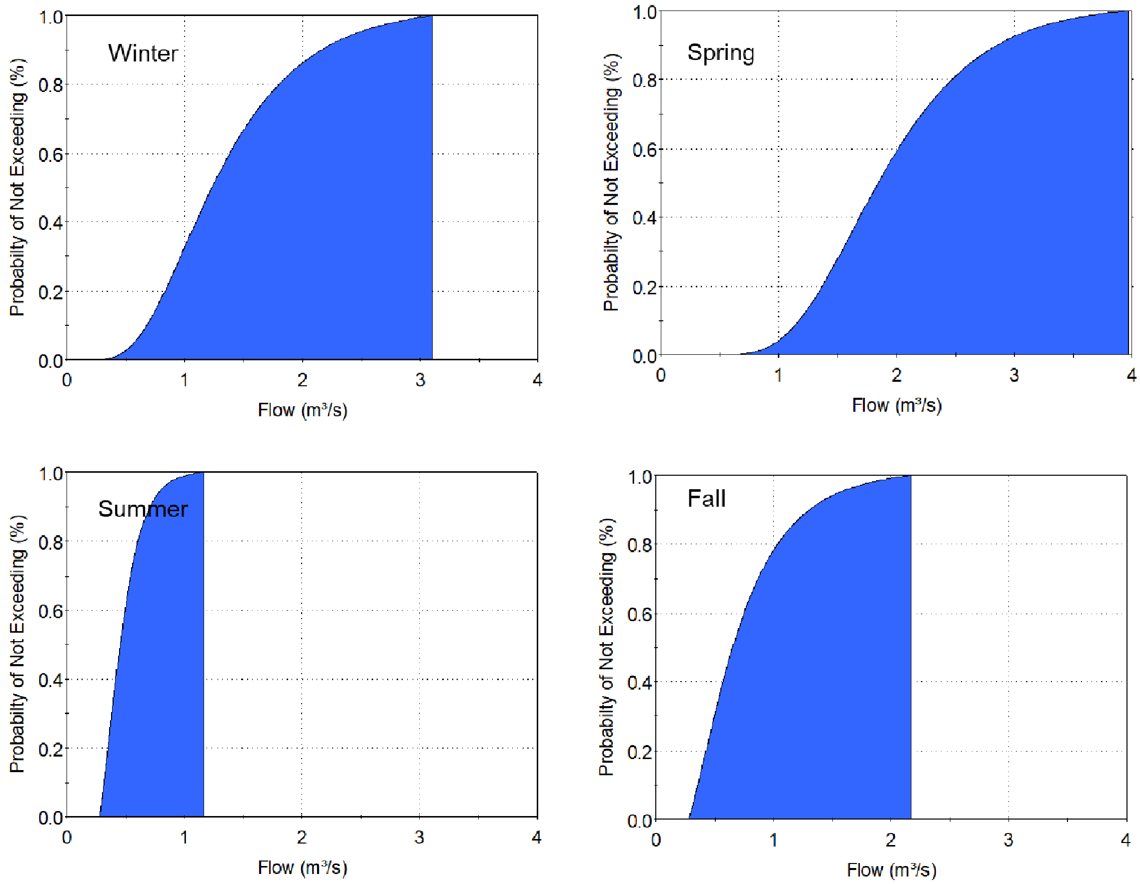


Figure 5: Seasonal Log-Normal Distribution of Flows in the Lyons Creek

3.1.2.3 Hydro Electric Power Canal (HEPC)

Table 14 shows the parameters associated with the log-normal distributions followed to characterize the flow in HEPC in GoldSim. Figure 6 shows the probability distribution of seasonal flow. The flow through Chippewa Creek was calculated based on the difference between the flow in the HEPC (input in GoldSim as per the distribution below) and the corresponding flow in Welland River East.

Table 14: Summary of Seasonal Flow Statistics for the Hydro Electric Power Canal

Parameter	Winter	Spring	Summer	Fall
Mean flow (m^3/s)	429	411	446	421
Maximum flow (m^3/s)	435	431	469	436
Minimum flow (m^3/s)	420	401	419	403
Standard deviation (m^3/s)	8.4	16.7	25.3	16.7

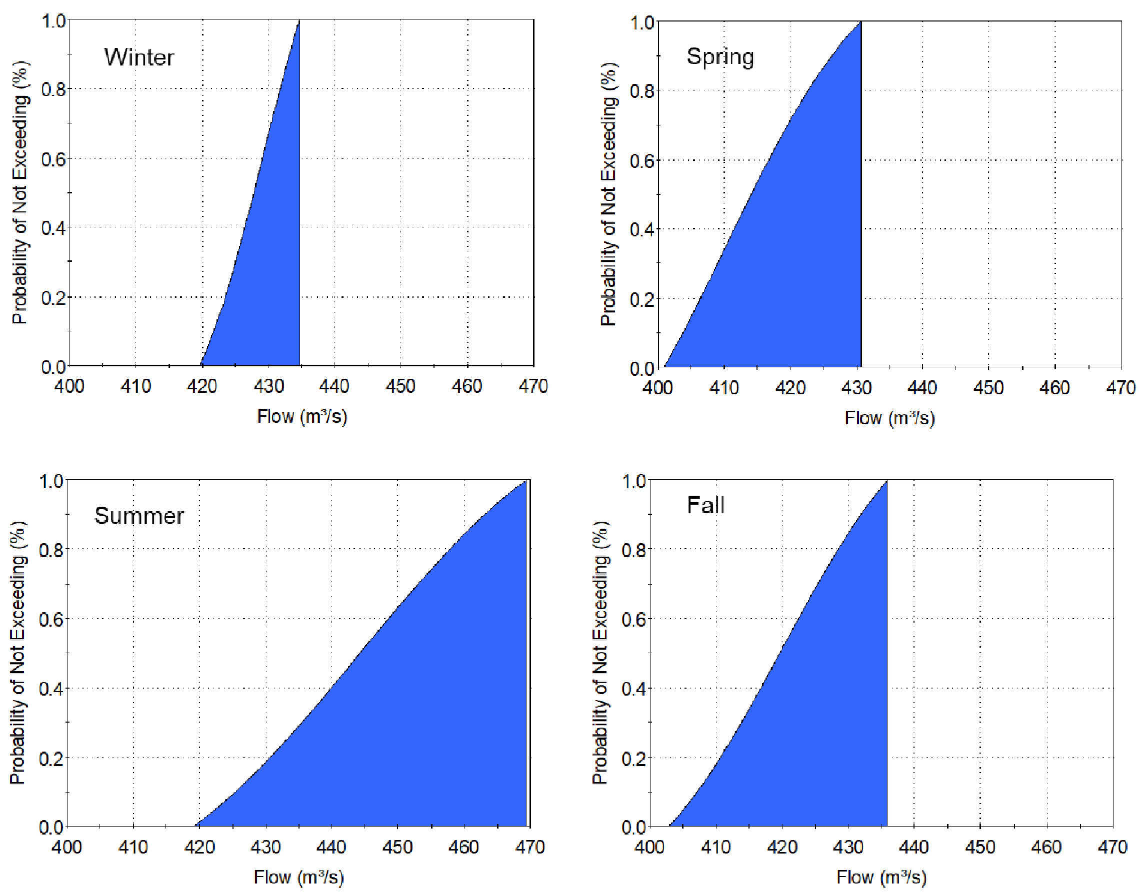


Figure 6: Seasonal Log-Normal Distribution of flows in the Hydro Electric Power Canal

3.1.2.4 Existing Niagara Falls Wastewater Treatment Plant

A statistical analysis of the flow data from the existing Niagara Falls WWTP showed little variation throughout the year. Table 15 shows the statistical flow distribution of existing Niagara Falls WWTP (based on data provided by Niagara Region), the flow limit based on existing ECA, and the assumed yearly mean flow used for modelling purposes in the GoldSim model.

Table 15: Summary of Seasonal Flow Statistics for Existing Niagara Falls Wastewater Treatment Plant, Environmental Compliance Approval Limit, and Assumed Mean Flow

Parameter	Winter	Spring	Summer	Fall	ECA Flow Limit	Assumed Mean Flow
Mean flow (m^3/s)	0.23	0.27	0.24	0.22	0.79[2]	0.47[3]
Minimum flow (m^3/s)	0.02	0.02	0.01	0.01	na[1]	na[1]
Maximum flow (m^3/s)	0.25	0.30	0.25	0.23	na[1]	na[1]
Standard deviation (m^3/s)	0.19	0.25	0.23	0.2	na[1]	na[1]

Notes

1. Mean flow which is assumed constant throughout the year (i.e., no probability distribution required).
2. Mean flow based on the ECA limit of 68,300 $m^{3/}$day.
3. Information provided by CIMA+.
4. Highlighted value corresponds with input to GoldSim model.

Given the above noted little variation throughout each season and between seasons, the mean value of 0.47 m^3/s was used to define the flow associated with the existing Niagara Falls WWTP. This fixed value was used instead of defining a probability distribution to characterize this input.

3.1.3 Model Validation

Model validation was done using the measured water quality data at the HEPC. The 75th percentile measurements at station PR001 was used for this purpose. Comparison were done considering two scenarios:

- excluding the CSOs from the model (No-CSO); and
- including the CSOs in the model (CSO).

The scenario that included the CSOs in the model also included, the overflow and secondary bypass from the existing Niagara Falls WWTP. As presented in Table 3, these flows represent approximately 94.0 to 99.6% of the total CSO flows. Water quality for each CSO (either overflow or secondary bypass) was allocated to each corresponding flow.

Table 16 compares the measured 75th percentile at PR001 with modelled (either CSO or No-CSO) 75th percentile concentration for the key parameters. These results show the effect of modelling CSO or No-CSOs does not affect the 75th percentile, which is to be expected given the low probability of occurrence of CSO events triggering high-load events...

Figure 7 though Figure 9 shows the box plots for comparing the measured and predicted concentration in the two scenarios as No-CSO and CSO for *E. coli*, total ammonia and phosphorus. These figures show how the consideration of CSOs in the model affects significantly the maximum modelled concentrations, specifically for *E. coli*.

When comparing the modelled results against the measured values, it is observed that total ammonia and *E. coli* are underpredicted by GoldSim. Generally, nitrate concentrations are well captured by GoldSim, with the later underpredicting winter concentrations by approximately 20%, and overpredicting nitrate concentrations for the rest of the year, with a maximum overestimation of 44% observed in fall. Phosphorus concentrations are also well captured in GoldSim, with general underprediction of phosphorus concentrations in winter and fall and overpredictions the rest of the year. The largest disagreement between measured and modelled concentration is observed in fall (23% underestimation) and spring (50% overprediction).

The differences between model predicted and measured concentrations are attributed to the following factors: exclusion of the variability of water quality in the model inputs, limited measured water quality data to better characterize chemistry in the system and exclusion of any other potential high-load sources which could affect water quality between the monitoring stations used to develop model inputs and monitoring station used to validate model output.

Table 16: Summary of GoldSim Model Verification

Parameter	Winter			Spring			Summer			Fall		
	PR001 Measured	Model without CSOs	Model with CSOs	PR001 Measured	Model without CSOs	Model with CSOs	PR001 Measured	Model without CSOs	Model with CSOs	PR001 Measured	Model without CSOs	Model with CSOs
Total Ammonia (mg/L)	0.18	0.05	0.05	0.38	0.07	0.07	0.25	0.05	0.05	0.28	0.05	0.05
E. coli (mg/L)	7,550	379	400	440	32	33	220	12	12	4,200	34	34
Nitrate (mg/L)	0.51	0.41	0.41	0.27	0.37	0.37	0.22	0.27	0.27	0.16	0.23	0.23
Total Phosphorus (mg/L)	0.059	0.049	0.049	0.018	0.036	0.036	0.020	0.024	0.025	0.040	0.031	0.032

Notes

1. All values in table are either measured or modelled 75th percentile concentrations.

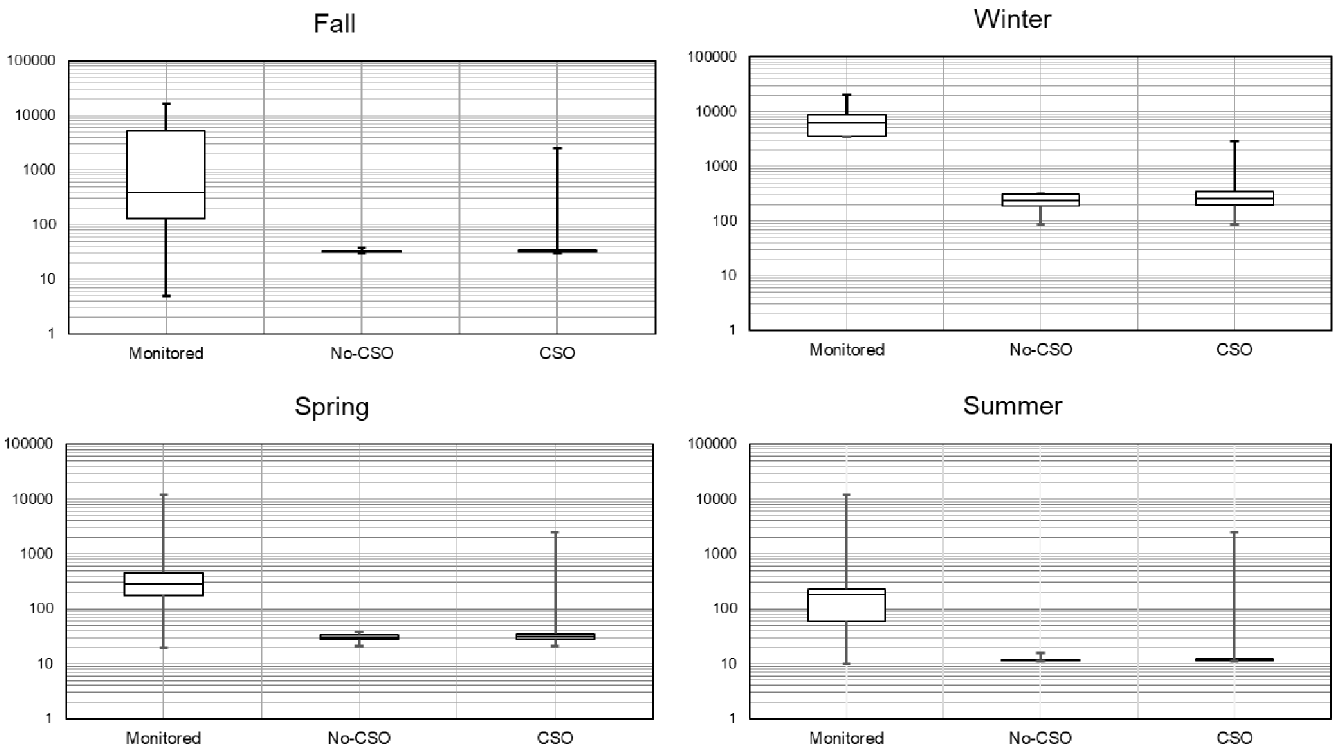


Figure 7: Box Plots Comparing Seasonal Measured and Modeled (No-CSO, CSO) *E. coli* Concentrations

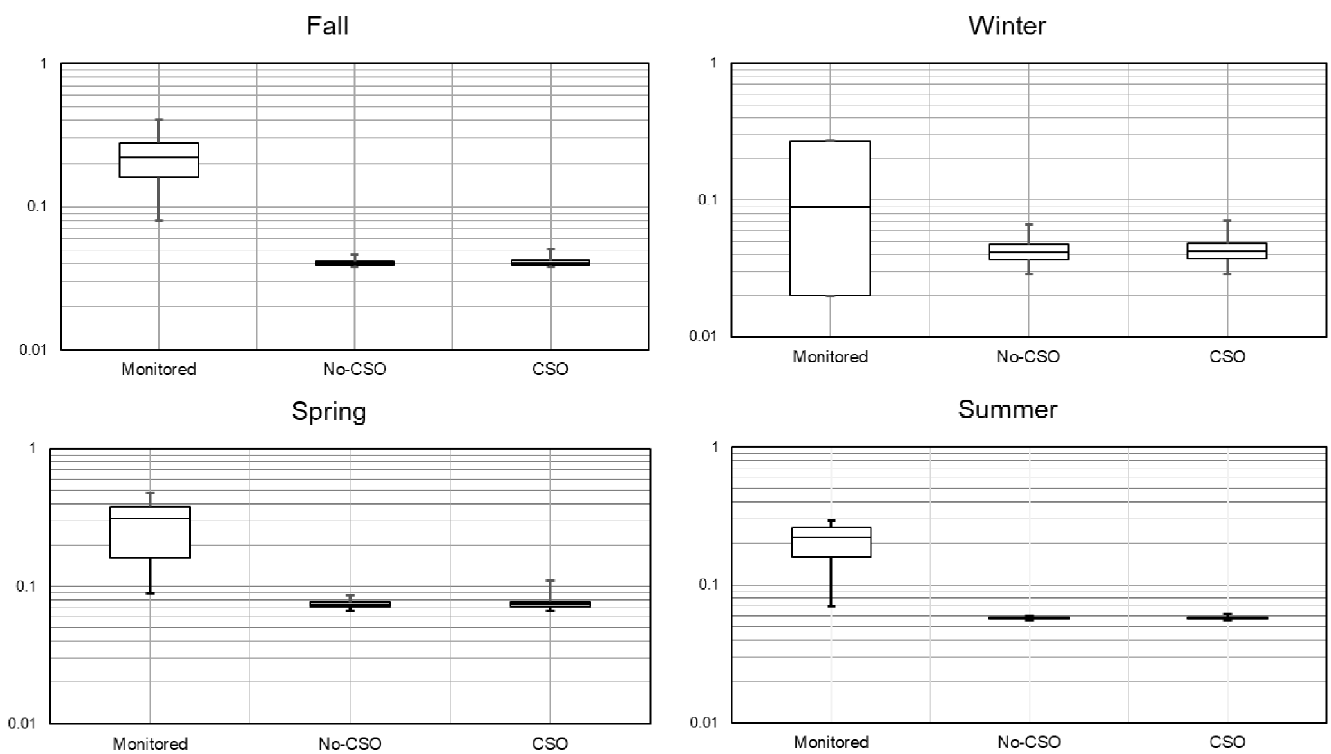


Figure 8: Box Plots Comparing Seasonal Measured and Modeled (No-CSO, CSO) Total Ammonia Concentrations

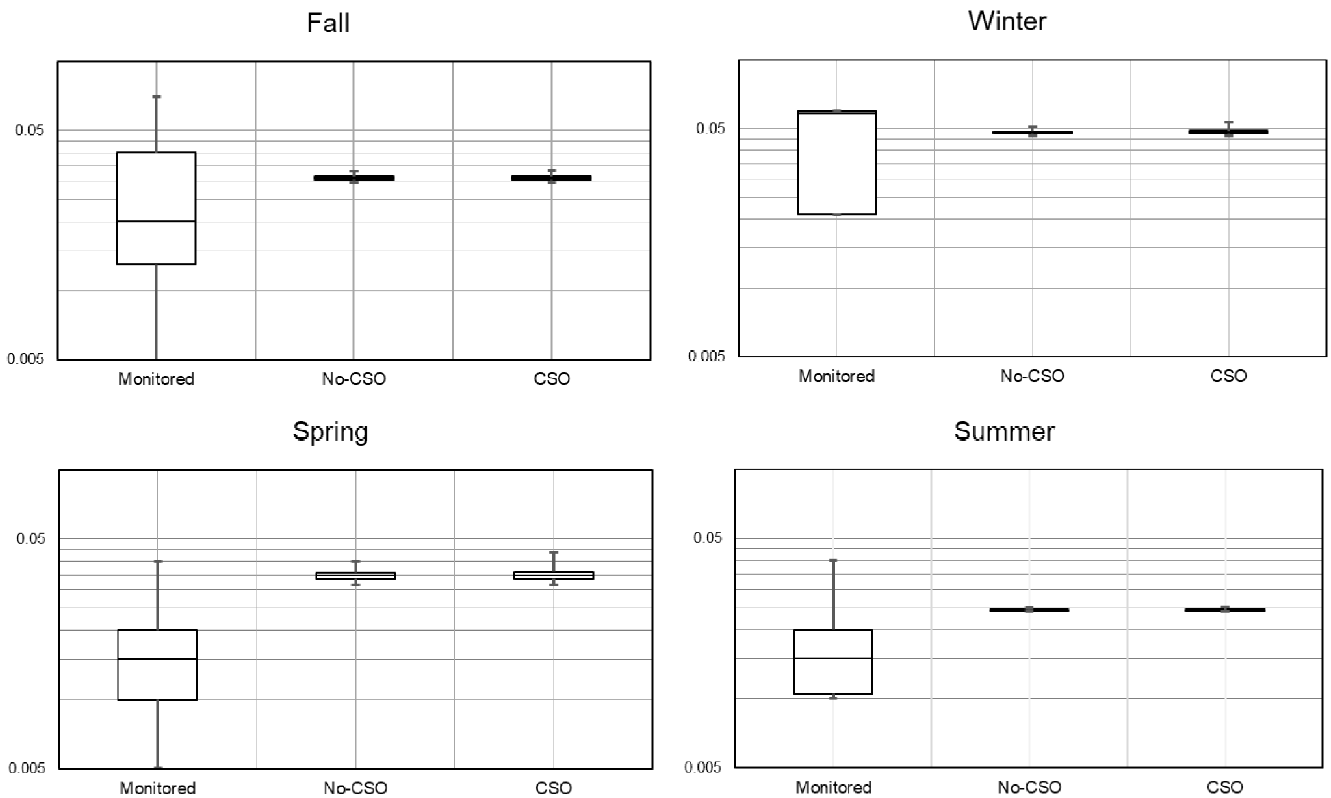


Figure 9: Box Plots Comparing Seasonal Measured and Modeled (No-CSO, CSO) Total Phosphorus Concentrations

3.1.4 Modelling Scenarios

Four different modelling scenarios were considered to assess assimilative capacity of the system under existing conditions, and under three potential locations of the new WWTP (Location 1 to Location 3). Each scenario was run independently for each season using a stochastic approach. These scenarios are described as follows:

- Baseline Scenario: To represent existing conditions, which includes the existing Niagara Falls WWTP but does not include the new WWTP.
- Scenario L1: Assumed the new WWTP discharges to the Welland River East, immediately upstream from Triangle Island.
- Scenario L2: Assumed the new WWTP discharges to the HEPC, downstream from Triangle Island and upstream from the existing Niagara Falls WWTP.
- Scenario L3: Assumed the new WWTP discharges to Chippewa Creek, immediately upstream from Triangle Island and downstream from the confluence with Lyons Creek.

3.1.5 Flow Implementation

As previously mentioned, the flow was implemented in the model based on the available data and the stochastic modelling using the GoldSim model for Welland River East, Lyons Creek and the HEPC. Flow in Chippewa Creek was estimated using the HEPC flow demand. The HEPC demand is provided by the flow coming from triangle west (Welland River East and the flow from new existing plant in case of Scenario L1) and flow coming from triangle east (Chippewa Creek, Lyons Creek and flow from new WWTP in case of scenario L2).
Therefore, flow in Chippewa Creek implemented in the model as the HEPC demand subtracted by flow coming from triangle west, Lyons Creek and L2. Flow from new WWTP was considered to be 0.347 m^3/s (30,000 m^3/d).

Effluent from the existing Niagara Falls WWTP was considered as per the average daily flow outlined in the ECA (i.e., 0.79 m^3/s equivalent to 68,300 m^3/day). CSOs associated with overflow and secondary bypass from the existing Niagara Falls WWTP were considered in this analysis.

3.1.6 Water Quality Implementation

The available data for water quality included ammonia, *E. coli*, nitrate, and total phosphorus. Water quality data associated with the 75th percentile was used for all inputs to the model with the exception of the effluent from the existing Niagara Falls WWTP, which considered water quality as per the ECA regulatory limits for total phosphorus and *E. coli*.

3.1.7 Water Quality Objectives

The allowable effluent concentration for the proposed WWTP were estimated by calculating the mass allowed in the system until reaching applicable water qualitive objectives. The threshold for *E. coli*, total phosphorus and nitrate were based on the guidelines provided in Table 4.

The GoldSim model does not incorporate accurate modelling of pH and water temperature. The fraction of the total ammonia that is unionized is a function of pH and temperature. The seasonal target values for total ammonia were back calculated from the PWQO limit of 0.0164 mg/L as nitrogen for unionized ammonia based on the monthly 75th percentile water temperature and pH in Chippewa Creek and the HEPC.

The seasonal thresholds for total ammonia, *E. coli*, nitrate and total phosphorus in the receiver used to estimate recommended effluent limits are summarized in Table 17.

Table 17: Summary of Water Quality Criteria used in GoldSim

Parameter	Winter	Spring	Summer	Fall
Total Ammonia (mg/L)[1]	1.150	0.288	0.142	0.176
E. coli. (cfu/100 mL)	100	100	100	100
Nitrate (mg/L)	3	3	3	3
Total Phosphorus (mg/L)	0.03	0.03	0.03	0.03

Note:

1. Total ammonia criteria based on target unionized ammonia concentration of 0.0164 mg/L as N and seasonal average water temperature and pH in receiving water.

3.1.8 Maximum Allowable Effluent Concentrations

The allowable mass modelled in the system was extracted for the local compliance point (immediate receiver where effluent from the new WWTP plant would enter the system) and at the system compliance point (downstream of the existing Niagara Falls WWTP). The recommended effluent concentrations were calculated by dividing the allowable mass by the flow from new WWTP. Large values in the table can be explained by the small flow rate in the proposed WWTP compared to the other flows in the system.

Table 18 shows the recommended effluent limits based on assimilative capacity at the local and system compliance points. These concentrations were calculated based on the GoldSim predictions for the 5% probability of exceedance.

These results show that the system is currently at capacity for *E. coli* in the summer and total phosphorus in the winter, spring, and fall.

The required effluent concentrations for total ammonia and total nitrate for the discharge into the Welland River East yielded the most restrictive treatment capacity, given the lower assimilative capacity of the immediate receiver. The differences between the discharges to the HEPC and Chippewa Creek are negligible in term required treatment.

Table 18: Summary of Maximum Allowable Effluent Concentrations from GoldSim Modelling

Parameter	Compliance Point	Winter			Spring			Summer			Fall		
		Location 1	Location 2	Location 3	Location 1	Location 2	Location 3	Location 1	Location 2	Location 3	Location 1	Location 2	Location 3
Total Ammonia (mg/L)	Local	24.5	1,347	1,312	0.7	262	261	nc	112	115	nc	157	159
	System	1,342			258			107			152		
E. coli (cfu/100 mL)	Local	nc	nc	55,235	nc	75,615	94,761	nc	107,736	107,869	nc	76,549	81,586
	System	nc			75,382			107,502			76,349		
Nitrate (mg/L)	Local	29	3,149	3,108	96	3,069	2,910	103	3,334	3,219	83	3,245	3,133
	System	3,142			3,062			3,328			3,238		
Total Phosphorus (mg/L)	Local	nc	nc	nc	nc	nc	3.28	nc	6.93	9.20	nc	nc	2.97
	System	nc			nc			6.28			nc		

Note:

1. “nc” denotes no capacity since existing background water quality exceeds applicable criteria (PWQO or CCME).

3.2 Mass Balance Modelling for Total Phosphorus, Ammonia, Nitrate, and *E. coli*

A secondary verification to the GoldSim model results, mass balance modelling was completed using 75th percentile background water quality concentrations and minimum supplemental flows. Mass balance modelling estimated the maximum allowable effluent concentrations for total phosphorus, E. coli, nitrate, total ammonia, CBOD5, and TSS and the minimum dissolved oxygen concentration. The mass balance models generally followed the same structure as the GoldSim model as shown on Figure 10 and provided seasonal estimates. One mass balance model was developed to assess total phosphorus, ammonia, nitrate, and *E. coli* such that both the local and system compliance points could be considered. Because dissolved oxygen and $CBOD_5$ are not independent, a specific mass balance model was developed for these two parameters simultaneously. A third mass balance model was developed for TSS since the water quality guideline for that parameter is based on an increase over ambient.

These models are intended to provide a secondary verification of the results provided by GoldSim by estimating the maximum allowable effluent concentrations for the worst-case conditions. The worst-case conditions were assumed to be the monthly cases where the low-flow conditions in each of the waterbodies occurred simultaneously.

The following points outline the inputs into the mass balance modelling:

- Total phosphorus, nitrate*, E. coli*, unionized ammonia, and TSS were modelled as conservative parameters and used the water quality limits provided in Table 4.
- The seasonal maximum allowable effluent concentrations for total ammonia were estimated based on the seasonal maximum allowable unionized ammonia concentration and 75th percentile values for water temperature and pH.
- The discharge of effluent from the existing Niagara Falls WWTP was assumed to be the rated capacity (68.3 MLD).
- The effluent discharge rate from the proposed WWTP was 30 MLD.
- Inflow concentrations from the Niagara River, Lyons Creek, and Welland River East were assumed to be equal to the 75th percentile of the measured seasonal concentrations.
- Where applicable, the existing effluent limits for the existing Niagara Falls WWTP were used (total phosphorus and *E. coli*).
- Since there are no effluent limits for the existing Niagara Falls WWTP for nitrate or ammonia, seasonal 75th percentile values based on measured data were used (Table 10).
- The effluent from both the existing Niagara Falls WWTP and the proposed plant was assumed to mix completely in the receiving water immediately after release.

Natural flows in the Welland River East were assumed to be negligible. The low-flow conditions in the Welland River East were assumed to be equal to the minimum supplemental flows from the Welland Canal as provided in Supplemental flows enter the Welland River East from the Welland Canal (St. Lawrence Seaway Management Corporation [SLSMC] 2019) as follows:

- A series of ports in the roof of the old syphon provide flow from the canal into the river. Depending on the season and water levels in the canal, the total flow ranges from 5 to 7 m^3/s.

- A pump at Port Robinson provides a flow of 0.97 m^3/s to a side channel of the Welland River East, which was cut-off from the main branch of the river during the straightening of the canal in the 1950s.
- The bypass of the Welland Water Treatment Plant provides a flow between the canal and the river that ranges from 4 m^3/s to 6 m^3/s.
- The effluent from the Welland Wastewater Treatment Plant provides a flow of 0.8 m^3/s (XCG 2007).

In general, the supplemental flows from the Welland Canal are from Lake Erie and have better water quality than that of the upstream areas of the Welland River.

Monthly estimates of the supplemental flows for the siphon ports, Port Robinson Pump, the Welland Water Treatment Plant and the Welland WWTP were provided by the SLSMC (SLSMC 2019) for the period 2014 to 2019 and are summarized in Table 1.

- Table 1Inflows from Lyons Creek were assumed to be equal to the pumping rates from the Welland Canal since naturally occurring low-flow conditions (e.g., 7Q20) are negligible (Section 2.1.4).
- Flows in the HEPC were assumed to be equal to the minimum daily average flow in the HEPC based on data provided by OPG between 2016 and 2018 (349 m^3/s).
- Flow in Chippewa Creek was assumed be the same as the flow in the HEPC less the contributions from the Welland River East and Lyons Creek.
- Seasonal maximum allowable effluent concentrations were estimated at local compliance point specific to each discharge location as well as at the system compliance point below the existing Niagara Falls WWTP.

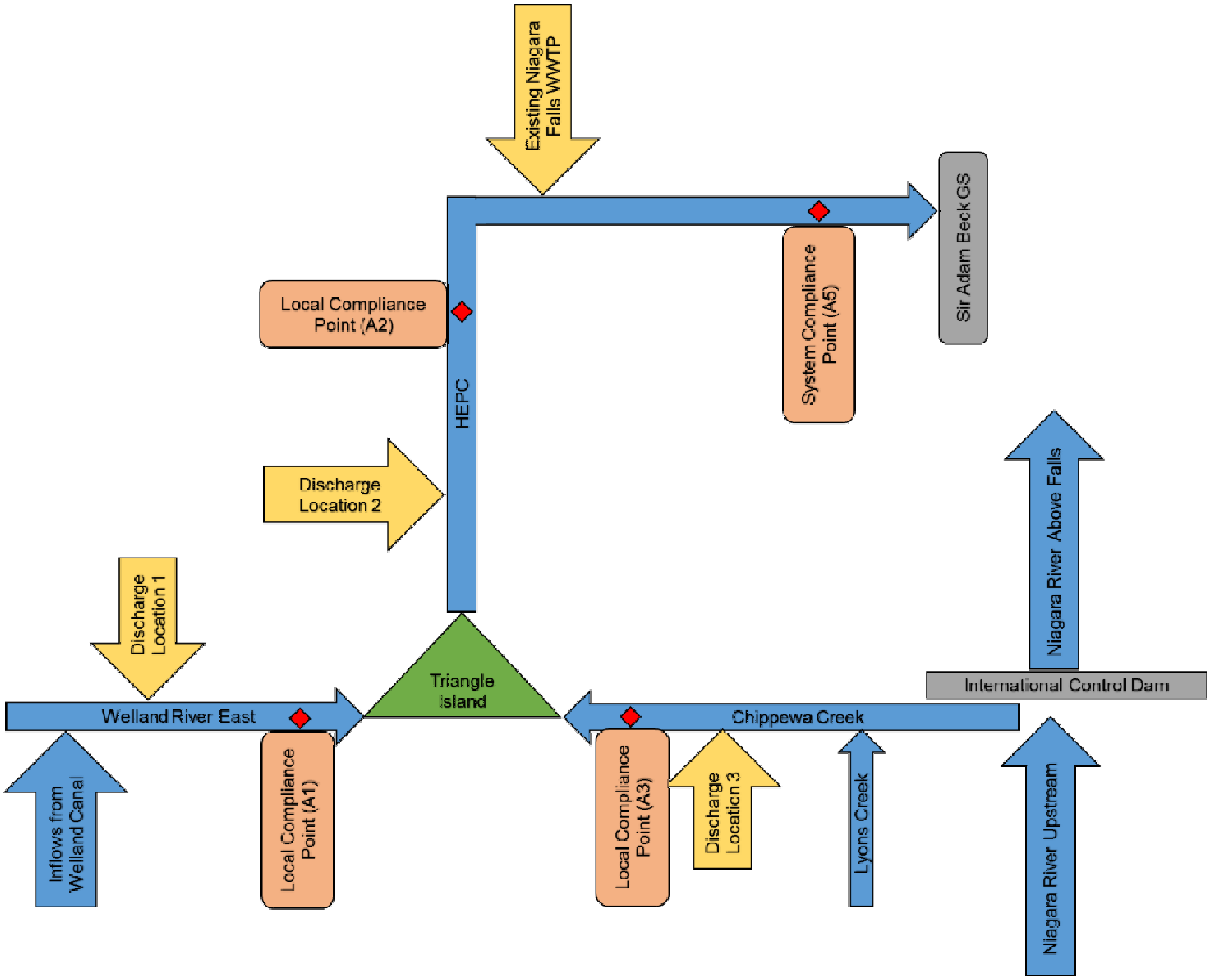


Figure 10: Schematic of Mass Balance Modelling for Total Phosphorus, Ammonia, Nitrate, and *E. coli*

The resulting estimates of the maximum allowable effluent concentrations are provided in Table 19. The modelling results suggest that:

- Poor water quality in the Welland River East provide no additional capacity for effluent in terms of total phosphorus and *E. coli* year-round and unionized ammonia during the summer.
- Elevated total phosphorus concentrations in the Niagara River during the winter are above the guideline and will limit capacity in Chippewa Creek and the HEPC.
- High *E. coli* contributions from the Welland River East limit the available capacity in the HEPC during the winter.
- High phosphorus loads from the Welland River East also limit the available capacity in the HEPC during the spring.
- Contributions from the existing Niagara Falls WWTP limit the available capacity at the system compliance point (A5) during the fall.

Table 19: Summary of Maximum Allowable Effluent Concentrations from Mass Balance Modelling of Worst Case Low-Flow Conditions

Parameter	Compliance Point	Winter			Spring			Summer			Fall		
		Location 1	Location 2	Location 3	Location 1	Location 2	Location 3	Location 1	Location 2	Location 3	Location 1	Location 2	Location 3
Unionized Ammonia (mg/L)	Local	0.5	15.5	15.0	0.4	15.3	15.0	nc	13.9	13.9	0.3	14.2	14.0
	System	15.5			15.3			13.8			14.2		
Total Ammonia (mg/L)	Local	33	1,227	1,194	4.4	284	280	nc	113	115	2.8	254	251
	System	1,216			610			101			243		
E. coli (cfu/100 mL)	Local	nc	nc	48,567	nc	78,132	85,459	nc	88,800	88,996	nc	69,113	71,728
	System	nc			78,132			88,800			69,113		
Nitrate (mg/L)	Local	23	2,644	2,621	67	2,681	2,614	99	2,750	2,652	73	2,807	2,735
	System	2,629			2,668			2,740			2,796		
Total Phosphorus (mg/L)	Local	nc	nc	nc	nc	nc	3.80	nc	5.69	7.65	nc	0.23	2.84
	System	nc			nc			5.02			nc		

Note:

1. "nc" denotes no capacity since existing water quality exceeds applicable criteria.

3.2.1 Comparison of Mass Balance Model Results to GoldSim Results

The following observations were made while comparing the results of the mass balance modelling to those of GoldSim:

- In cases where both models predicted assimilative capacity, the results from the mass balance model were lower than the results of GoldSim. This was expected since the mass balance model assumed the worst-case conditions (e.g., all low flows occur at once), which is expected to occur less than 5% of the time in the GoldSim model.
- With only one exception, both models predicted no assimilative capacity for the same cases.
- In the case for a discharge into the HEPC during the fall, GoldSim predicts no capacity for total phosphorus, while the mass balance model estimates a maximum allowable effluent concentration of 0.23 mg/L. Further investigation indicates that the difference is attributed to phosphorus loads from Welland River East. The mass balance model assumes that natural flows in Welland River East are negligible, while GoldSim uses a distribution of flows that include some natural flows. This results in a lower total phosphorus load in the mass balance model compared to that in GoldSim. Sensitivity analysis using the mass balance model suggest that natural flows from Welland River East were as low as 2 m^3/s increase the total phosphorus loads to the HEPC enough to eliminate any assimilative capacity in the fall.

3.3 Modelling for Niagara River Discharge (Location 4)

The following points summarize the approach used to assess the discharge to the Niagara River (Location 4):

- This discharge was assessed as a single port outfall (e.g., pipe) into a wide shallow river.
- The compliance point was assumed to be at the top of Niagara Falls along the Canadian shore approximately 1.6 km downstream of the ICD.
- The low-flow condition over the falls was assumed to be the minimum regulated daily average flow over the falls as outlined in the Niagara Treaty (2,242 m^3/s during the tourist season and 2,124 m^3/s during the non-tourist season). These flow conditions are the result of the operation of the ICD.
- The discharge location was assumed to be below the ICD and as such, water level fluctuations in the Grass Island Pool due to the operation of the ICD are not expected to affect the mixing of the effluent in the Niagara River.
- Since neither bathymetric data or current measurements are available for the Niagara River below the ICD, hydraulic modelling was completed to estimate the depth and current speed in that section of the Niagara River (see Section 3.3.1).
- Given that the Niagara River below the ICD is fast moving and wide, complete mixing with the effluent into the Niagara River flow cannot be expected before the compliance point. A Gaussian Plume model was used to estimate the width if the effluent plume at the compliance point to approximate the amount of river flow available for effluent dilution before passing the compliance point (See Section 3.3.2).
- Maximum allowable effluent concentrations were estimated for each season based on the available flow for dilution, upstream water quality, and ambient water temperature and pH.

3.3.1 Estimation of Hydraulic Conditions

Manning equation (Manning 1891) was iteratively solved to estimate the flow depth and current speed:

$$Q = UBH = \frac{1}{n}\left(\frac{BH}{B+2H}\right)^{2/3} S^{1/2}$$

Where:
- Q total flow in river (m^3/s),
- U current speed (m/s),
- B river width (m),
- H depth (m),
- n Manning's roughness coefficient, and
- S slope of river (m/m).

For this assessment, the average river width was assumed to be 887 m based on four width measurements (Google Earth) and the Manning's Roughness Coefficient was assumed to be 0.03.

The slope of the Niagara River was based on a downstream distance of 1,600 m and a reported river drop of 15 m between the ICD and the falls (Niagara Parks 2018). The slope for this section of the Niagara River was estimated to be 0.009 (0.9%).

The estimated low-flow hydraulic conditions in the Niagara River below the ICD for tourist and non-tourist periods are summarized in Table 20. For both periods, the estimated water depths are less than 1 m and the current speeds are greater than 2.8 m/s. Under these conditions, the effluent is expected to travel from the discharge location to the compliance point in less than 10 minutes.

Table 20: Summary of Estimated Low-Flow Hydraulic Conditions in Niagara River below the ICD

	Non-Tourist Season Winter Regulated Minimum Flow Over Falls	Tourist Season Spring/Summer/Fall Regulated Minimum Flow Over Falls
Flow over Falls (m^3/s)	2,124	2,242
Average Width (m)	887	
Depth (m)[1]	0.87	0.85
Current Speed (m/s)[1]	2.89	2.83
Lateral Dispersion Coefficient (m^2/s)[2]	0.146	0.139

Note:
1. Estimated using Manning's Equation.
2. Estimated using equations from Fischer (1979).

3.3.2 Gaussian Plume Modelling

A 2-dimensional Gaussian plume model is used to estimate the spread of the effluent in the Niagara River for the conditions provided in Table 20. The general form of a Gaussian plume for a continuous release from a shoreline discharge is:

$$C(x,y) = \frac{2W}{H\sqrt{4\pi D_y Ux}} e^{\left(-Uy^2/_{4D_y x}\right)}$$

Where:

Where:	C(x,y)	predicted concentration at specified location (g/m³),
	x	downstream distance (m),
	y	distance from shoreline (m),
	W	effluent mass loading rate (flow x concentration) (g/s),
	U	current speed (m/s),
	H	depth (m), and
	D_y	lateral dispersion coefficient (m^2/s).

The lateral dispersion coefficient was estimated as follows (Fischer et al. 1979):

$$D_y = 0.6HU^*$$

$$U^* = \sqrt{gHS}$$

Where:	U*	shear velocity (m/s),
	g	acceleration due to gravity (m/s^2), and
	S	river slope (m/m)

Based on the Gaussian plume modelling, at a distance of 1,600 m the width of plume that contains 95% of the effluent is predicted to be approximately 25 m or approximately 3% of the average river width. This suggests that the effluent will only mix with 3% of the total flow in the Niagara River below the ICD. This translates to available river flows for dilution of 72.7 m^3/s during the tourist season and 63.7 m^3/s during the non-tourist season.

3.3.3 Maximum Allowable Effluent Concentrations

A mass balance model was used to estimate the seasonal maximum allowable effluent concentrations for the Niagara River discharge option based on seasonal upstream water quality. For parameters listed in the ECA, the 75th percentile was used for the upstream water quality while for water temperature and pH seasonal averages were used.

Seasonal low-flow conditions were based on the minimum daily average flow requirements from the Niagara Treaty that occur in each of the assessment seasons. The mass balance assumed an effluent flow rate of 30 MLD (0.35 m³/s).

The maximum allowable effluent concentration was estimated for each parameter (except total ammonia) and season using:

$$C_e = \frac{(Q_e + Q_r)C_g - Q_r C_r}{Q_e}$$

Where: C_e allowable effluent concentration (mg/L),
C_r river/background concentration (mg/L),
C_g water quality guideline/target (mg/L),
Q_r upstream river flow (m³/s), and
Q_e effluent flow rate (m³/s)

The maximum allowable total ammonia concentrations were based on the maximum allowable unionized ammonia concentrations, average seasonal water temperature, and average seasonal pH.

A summary of the mass balance modelling and the resulting maximum allowable effluent concentrations are provided in Table 21.

Table 21: Detailed Summary of Allowable Effluent Concentrations for Discharge to Niagara River

	Winter	Spring	Summer	Fall
Flow Conditions				
Total Flow Over Falls (m^3/s)	2,124	2,124	2,424	2,124
Flow Available for Dilution (m^3/s)	63.7	63.7	72.7	63.7
Effluent Flow	0.347	0.347	0.347	0.347
Ultimate Dilution	185:1	185:1	210:1	185:1
Total Phosphorus				
Background / Upstream Concentration (mg/L)	0.043	0.026	0.022	0.027
PWQO / Target at Flow over Falls (mg/L)	0.030	0.030	0.030	0.030
Allowable Effluent Concentration (mg/L)	No Capacity	0.764	1.705	0.581
Nitrate				
Background / Upstream Concentration (mg/L)	0.310	0.310	0.260	0.180
PWQO / Target at Flow over Falls (mg/L)	3.0	3.0	3.0	3.0
Allowable Effluent Concentration (mg/L)	497	497	577	521
E. coli				
Background / Upstream Concentration (cfu/100 mL)	50	12	8	26
PWQO / Target at Flow over Falls (cfu/100 mL)	100	100	100	100
Allowable Effluent Concentration (cfu/100 mL)	9,276	16,249	19,368	13,680
Unionized and Total Ammonia				
75th Percentile Water Temperature (ºC)	2.5	10.1	23.9	20.1
75th Percentile pH	8.1	8.2	8.3	8.2
Fraction Unionized Ammonia (%)	1.32%	2.88%	10.09%	5.95%
Upstream Total Ammonia Concentration (mg/L)	0.014	0.046	0.044	0.032
Upstream Unionized Ammonia Concentration (mg/L)	0.00018	0.00133	0.00444	0.00190
PWQO / Target at Flow over Falls (mg/L)	0.0164	0.0164	0.0164	0.0164
Allowable Effluent Unionized Ammonia Concentration (mg/L)	2.99	2.78	2.52	2.68
Allowable Effluent Total Ammonia Concentration (mg/L)	227	97	25.0	45

3.4 Mass Balance Modelling for Dissolved Oxygen, $CBOD_5$, and Total Suspended Solids

Allowable effluent concentrations were estimated for dissolved oxygen, CBOD5, and TSS using a spreadsheet-based mass-balance model. These parameters could not be modelled in GoldSim for the following reasons:

- dissolved oxygen and $CBOD_5$ are interconnected such that they could not be represented in GoldSim and,
- the criteria for TSS (see Section 2.2.1) is based on an increase over background.

The mass balance modelling was based on low flow conditions that represent the minimum regulated flows over the falls (Section 2.1.3.2), supplemental inflows in the Welland River (Section 0), and estimated 7Q20 flows in the HEPC (Section 2.1.5). For the discharge to the Niagara River, the available flow for dilution was assumed to be 3% of the total flow over the falls (Section 3.3.2). A summary of the flows used in the mass balance modelling for dissolved oxygen, $CBOD_5$, and TSS is provided in Table 22.

Table 22: Summary of Flows Used in Mass Balance Modelling

Season	Niagara River Below ICD		Chippewa Creek[3] (m^3/s)	Welland River East[4] (m^3/s)	HEPC[5] (m^3/s)
	Total[1] (m^3/s)	Available for Dilution[2] (m^3/s)			
Winter	2,124	63.7	338	11.4	349
Spring	2,142	63.7	337	12.2	349
Summer	2,224	67.3	335	13.6	349
Fall	2,124	63.7	336	13.0	349

Notes:

1. Minimum flows as defined in Niagara Treaty of 1950.
2. Only 3% of flow available for dilution before reaching falls (Section 3.3.2).
3. Flow in HEPC less flow from Welland River East.
4. Sum of all supplemental flows into Welland River East from Welland Canal.
5. Low flow condition (7Q20) for flow in HEPC.

3.4.1 Dissolved Oxygen and $CBOD_5$

Since dissolved oxygen and $CBOD_5$ of the effluent and background water all affect the downstream dissolved oxygen concentrations, these two parameters must me assessed together. The downstream dissolved oxygen at any downstream location is determined by the mixed (effluent and river) concentration of dissolved oxygen and the amount of oxygen consumed by the $CBOD_5$ in the time taken to reach that location. Other factors that affect the downstream dissolved oxygen include surface reaeration and algal growth/decay.

The assessment of dissolved oxygen and $CBOD_5$ provides a conservative estimate of allowable effluent concentrations based on the following assumptions:

- Although measurements of dissolved oxygen in the Niagara River and HEPC are frequently at or above saturation due to turbulent flow conditions that provide a high degree of surface reaeration, surface reaeration is not included in this assessment.
- Given the typical clarity of the water in the study area, the effects of algae are assumed to be negligible and are not included in the assessment.

- Given the short retention time in the system (e.g., less than a few hours), it is expected that only a fraction of the $CBOD_5$ will be consumed before leaving the study area. This assessment assumes that 50% of the $CBOD_5$ from upstream sources and the effluent will be consumed before leaving the system.
- $CBOD_5$ data was not available for the Niagara River. As such a background $CBOD_5$ concentration of 2 mg/L was assumed based on the highest seasonal 75th percentile $CBOD_5$ concentration found for the Welland River East (Table 5). These upstream conditions were applicable to the discharges into Chippewa Creek and the Niagara River.
- Upstream $CBOD_5$ concentrations in the Welland River East were based on the seasonal 75th percentile of the measured data.
- Upstream dissolved oxygen concentrations were based on the seasonal 25th percentile of the measured data.
- Upstream $CBOD_5$ and dissolved oxygen for the HEPC discharge were based on flow weighted values for Chippewa Creek and Welland River East.
- Water temperatures (required to estimate dissolved oxygen saturation concentrations) were based on the seasonal 75th percentile temperature values for Chippewa Creek, the HEPC, and Welland River East.
- Given the high degree of surface reaeration in the HEPC, dissolved oxygen and $CBOD_5$ were not assessed at the system compliance point (Sir Adam Beck GS).
- The assessment was based on the dissolved oxygen criteria for warm water fisheries (47% of saturation below 20ºC and 4 mg/L above 20ºC).

The allowable effluent $CBOD_5$ concentration was estimated by re-arranging the following equation:

$$Q_d D_d = Q_r D_r - f Q_r B_r + Q_e D_e - f Q_e B_e$$

Where:	Q_d	downstream flow (m³/s) equal to sum of upstream and effluent flows,
	Q_r	upstream flow (m³/s),
	Q_e	effluent flow (m³/s),
	D_d	downstream dissolved oxygen concentration (mg/L) equal to guideline,
	D_r	upstream dissolved oxygen concentration (mg/L),
	D_e	effluent dissolved oxygen concentration (mg/L),
	B_r	upstream $CBOD_5$ concentration (mg/L),
	B_e	effluent $CBOD_5$ concentration (mg/L), and
	f	fraction of $CBOD_5$ consumed in study area (assumed to be 0.5).

Estimates of the allowable seasonal effluent $CBOD_5$ concentrations are provided in Table 23 for three levels of effluent dissolved oxygen saturation (10%, 50%, and 90%). Allowable concentrations for $CBOD_5$ are all greater than the minimum standard limit for secondary treated effluent of 15 mg/L.

The results indicate that allowable $CBOD_5$ concentrations are not sensitive to the dissolved oxygen levels in the effluent. Therefore, effluent dissolved oxygen concentration equal to 50% of the saturation concentration is recommended. The corresponding allowable seasonal effluent $CBOD_5$ concentrations will be carried forward in this assessment.

Table 23: Estimated Allowable $CBOD_5$ Concentrations Based on Effluent Dissolved Oxygen

Discharge Location	Season	Allowable Effluent $CBOD_5$ Concentration		
		Eff DO = 10% Sat[1]	Eff DO = 50% Sat[1]	Eff DO = 90% Sat[1]
Welland River East (Location 1)	Winter	360	371	382
	Spring	376	384	392
	Summer	239	**245**	252
	Fall	282	289	296
HEPC (Location 2)	Winter	6,758	6,768	6,779
	Spring	6,793	6,800	6,808
	Summer	7,934	7,940	7,947
	Fall	5,943	**5,952**	5,960
Chippewa Creek (Location 3)	Winter	6,370	6,380	6,391
	Spring	6,376	6,384	6,391
	Summer	7,682	7,689	7,695
	Fall	5,699	**5,707**	5,715
Niagara River (Location 4)	Winter	1,194	1,204	1,215
	Spring	1,201	1,275	1,283
	Summer	1,536	1,461	1,468
	Fall	1,074	**1,083**	1,091

Note:

1. Dissolved oxygen concentration in effluent expressed as percent of saturation.
2. **Bold** values indicate maximum allowable effluent concentrations carried forward in assessment.

3.4.2 Total Suspended Solids

The assessment of TSS was based on the following assumptions:

- Upstream TSS concentrations in the Welland River East were based on the seasonal 75th percentile of the measured data.
- Upstream TSS concentrations in the Niagara River, Chippewa Creek, and the HEPC were based on an annual 75th percentile of the measured data in the Niagara River (11.3 mg/L).

The allowable effluent TSS concentration was estimated by re-arranging the following equation:

$$(Q_r + Q_e)(C_r + \Delta C) = Q_r C_r + Q_e C_e$$

Where:
Q_r upstream flow (m^3/s),
Q_e effluent flow (m^3/s),
C_r upstream TSS (mg/L),
C_e effluent TSS (mg/L), and
ΔC allowable TSS concentration increase (5 mg/L).

The estimated allowable seasonal effluent concentrations for TSS are provided in Table 24 and indicate that the allowable effluent TSS concentration show little seasonal variation. Allowable concentrations for TSS are all greater than the minimum standard limit for secondary treated effluent of 15 mg/L.

Table 24: Estimated Allowable Seasonal Effluent TSS Concentrations

Discharge Location	Season	Allowable Total Suspended Solids (mg/L)
Welland River East (Location 1)	Winter	204
	Spring	202
	Summer	213
	Fall	**201**
HEPC (Location 2)	Winter	5,047
	Spring	5,047
	Summer	**5,046**
	Fall	**5,046**
Chippewa Creek (Location 3)	Winter	4,880
	Spring	4,866
	Summer	**4,846**
	Fall	4,855
Niagara River (Location 4)	Winter	934
	Spring	985
	Summer	**934**
	Fall	**934**

Note:

1. **Bold** values indicate maximum allowable effluent concentrations carried forward in assessment.

4.0 DERIVATION OF RECOMMENDED EFFLUENT LIMITS

The following sections outline the development of the recommended effluent limits and limits based on the ACS and include the following details for each discharge location:

- the applicable water quality assessment points for each discharge location alternative,
- if specific parameters meet or exceed relevant criteria and whether a Policy 2 Condition applies,
- the critical season for each parameter and location, and
- an appropriate treatment technology for the location.

A quick summary of the adopted approach is provided below. Using this approach, the detailed evaluation of assimilative capacity and selection of treatment technologies is documented for each discharge location alternative in Section 4.1 through 4.4.

Water Quality Assessment Points

The water quality effects of introducing the new WWTP at each of four discharge location alternatives is evaluated at selected downstream assessment points. Referring to Section 0, the new WWTP effluent at each discharge location alternative is specifically evaluated at local assessment points (A1, A2, A3 or A4), located immediately downstream of each discharge location alternative, and at a system assessment point (A5) in the HEPC below the existing Niagara Falls WWTP (Locations 1, 2, and 3 only).

Available Assimilative Capacity

The available assimilative capacity for each assessment point is first considered without the effluent inputs from the new WWTP to determine if there is any for each of the parameters at the local compliance point. Where locations are shown to have capacity to assimilate effluent, a treatment technology was selected that could meet the maximum allowable effluent concentrations for each parameter. In cases were there was no available assimilative capacity (e.g., Policy 2), the effluent quality was selected such that the effluent concentration would be equal or less than the existing background conditions.

The typical effluent quality for the available treatment technologies considered in this study, based on information available from the MECP (MECP 2019), are summarized in Table 25.

Table 25: Typical Effluent Quality for Various Treatment Processes

Process	Effluent Parameter[1,2]			
	$CBOD_5$ (mg/L)	Total Suspended Solids (mg/L)	Total Phosphorus (mg/L)	Total Ammonia (mg/L as N)[3]
Conventional Activated Sludge System				
Without Phosphorus Removal	25	25	3.5	15 to 20
With Phosphorus Removal	25	25	<1.0	15 to 20
With Phosphorus Removal and Filtration	10	10	0.3	15 to 20
With Nitrification and Phosphorus Removal	25	25	<1.0	<3
Membrane Bioreactor				
Without Phosphorus Removal	2	1	3.0	15 – 20
With Phosphorus Removal	2	1	0.1	15 – 20
With Phosphorus Removal and Filtration	2	1	0.1	0.3

Notes:

1. Taken from "Design Considerations for Sewage Treatment Plants" (MECP 2019)
2. The above values are based on raw sewage with CBOD5 = 150-200 mg/L, Soluble CBOD5 = 50% of CBOD5, TSS = 150-200 mg/L, TP = 6-8 mg/L, TKN = 30-40 mg/L, TAN = 20-25 mg/L.
3. TAN (total ammonia nitrogen) concentrations may be lower during warm weather conditions if nitrification occurs.

With regard to parameters not listed in Table 25, the following assumptions have been used:

- Any treatment plant with disinfection can expect to have an *E. coli* concentration objective of less than 200 cfu/100 mL,
- If needed, aeration of the dissolved oxygen concentration in the final effluent can be provided to at least 80% of the saturation concentration.
- The expected effluent nitrate concentration from an activated sludge system without denitrification was assumed to be 20 mg/L.

4.1 Location 1 – Welland River East

4.1.1 Overview of Existing Conditions

The Welland East discharge would release effluent to Welland River East between Montrose Road and Triangle Island. Under normal conditions, the effluent is expected to travel downstream into the HEPC and eventually enter the Niagara River at the Sir Adam Beck GS. The local compliance point (A1), in the Welland River East just upstream of Triangle Island, and the system compliance point (A5), in the HEPC below the existing Niagara Falls WWTP (both shown on Figure 11).

The Welland River East discharge is not expected to affect water quality in Chippewa Creek or in the Niagara River upstream of the Sir Adam Beck GS.

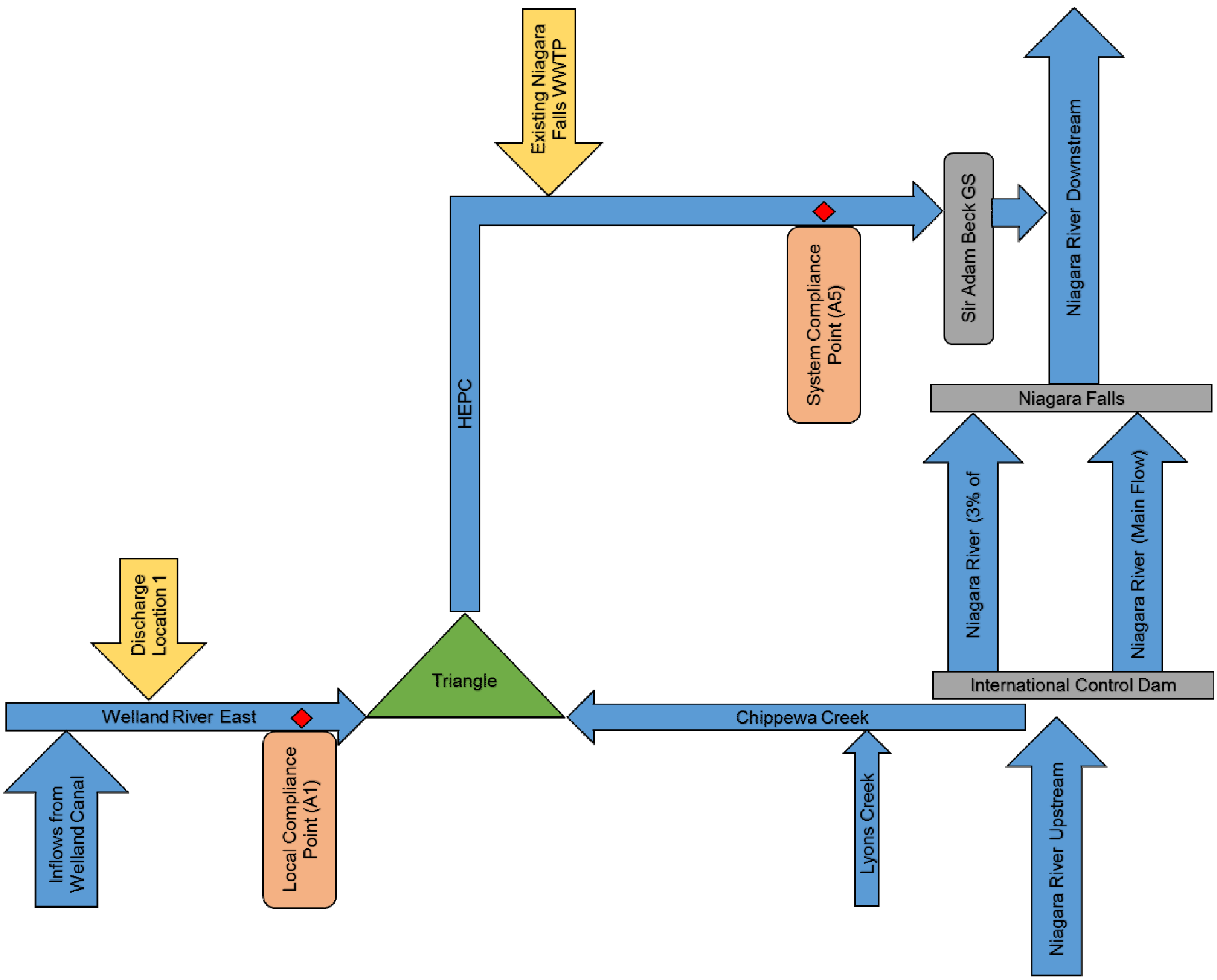


Figure 11: Local and System Compliance Points for Discharge at Location 1 – Welland River East

4.1.2 Phosphorus

The total phosphorus concentrations in the Welland River East are elevated and consistently exceed the applicable PWQO (0.03 mg/L). The seasonal geometric mean concentration ranges from 0.04 mg/L to 0.12 mg/L while the 75th percentile concentrations range from 0.06 mg/L to 0.14 mg/L. Total phosphorus concentrations are typically higher at Welland (WR010) than at Montrose Road (WR011). It is suspected that the water quality at Montrose Road is periodically affected flow reversals that occur due to the operation of the ICD (e.g., water from the Niagara River with better water quality is periodically samples at WR011).

The predicted maximum allowable effluent concentrations for phosphorus are presented in Table 26. The elevated upstream total phosphorus concentrations result in Policy 2 conditions year-round at the local and system compliance points. Discharge from the existing Niagara Falls WWTP results in no additional capacity to receive phosphorus at the system compliance point in all seasons except summer.

Table 26: Maximum Allowable Seasonal Total Phosphorus Concentrations for Discharge at Location 1 – Welland River East

Season	Upstream[1] (mg/L)	Maximum Allowable Effluent Concentration (mg/L)	
		Local Compliance Point	System Compliance Point
Winter	0.140	No Capacity[2]	No Capacity[2]
Spring	0.160		
Summer	0.080		
Fall	0.100		

Notes:

1. 75th percentile of seasonal upstream concentrations.
2. No capacity due to elevated concentrations at the compliance point.

Since the upstream phosphorus concentration in Welland River East exceed the PWQO (0.03 mg/L), it is considered a Policy 2 receiver with respect to total phosphorus. As such, the effluent concentration is not to exceed background conditions. The seasonal 75th percentile phosphorus concentration varies from 0.075 mg/L to 0.125 mg/L. It is recommended that the annual average 75th percentile value be used (0.10 mg/L) as the effluent limit for phosphorus.

Based on the information provided in Table 25, in terms of total phosphorus discharge the recommended treatment technology at Location 1 is equivalent to a membrane bioreactor with phosphorus removal.

4.1.3 Nitrate

The seasonal geometric mean nitrate concentration ranges from 0.33 mg/L to 2.32 mg/L while the 75th percentile concentrations range from 0.48 mg/L to 2.38 mg/L. The highest nitrate concentrations, which typically occur during the winter, are approaching the CCME guideline (3 mg/L). This suggests that there may be seasonal limitations on the maximum allowable effluent concentration of nitrate.

The predicted maximum allowable effluent concentrations for nitrate are presented in Table 27. In general, the local compliance point provides the most restrictive conditions. Based on the modelling results, the most restrictive value is 29 mg/L.

Based on the assumptions in Section 4.0, a conventional activated sludge system without denitrification is expected to provide effluent nitrate concentrations of 20 mg/L. As a result, nitrate limits would not be required for Location 1.

Table 27: Maximum Allowable Seasonal Nitrate Concentrations for Discharge at Location 1 – Welland River East

Season	Upstream[1] (mg/L)	Maximum Allowable Effluent Concentration (mg/L)	
		Local Compliance Point	System Compliance Point
Winter	2.38	**29 (23)**	3,142 (2,629)
Spring	1.11	96 (67)	3,062 (2,668)
Summer	0.49	103 (99)	3,328 (2,740)
Fall	1.05	83 (73)	3,238 (2,796)

Notes:

1. 75th percentile of seasonal upstream concentrations.
2. Values in brackets refer to predictions from the mass balance modelling approach, if different from the GoldSim modelling approach.

4.1.4 Ammonia

The seasonal geometric mean total ammonia concentration ranges from 0.07 mg/L to 0.47 mg/L while the 75th percentile concentrations range from 0.09 mg/L to 0.59 mg/L. The corresponding unionized ammonia concentrations are below the applicable PWQO (0.0164 mg/L as N) for all the seasons except summer.

The maximum allowable effluent concentrations for total and unionized ammonia are presented in Table 28. In general, the local compliance point provides the most restrictive conditions. The elevated upstream unionized ammonia concentrations result in Policy 2 conditions in the summer.

Table 28: Maximum Allowable Seasonal Total and Unionized Ammonia Concentrations for Discharge at Location 1 – Welland River East

Season	Total Ammonia			Unionized Ammonia		
	Upstream	Maximum Allowable Concentration (mg/L)		Upstream	Maximum Allowable Concentration (mg/L)	
		Local Compliance Point	System Compliance Point		Local Compliance Point	System Compliance Point
Winter	0.59	25 (33)	1,342 (1,216)	0.001	0.3 (0.5)	12.5 (15.5)
Spring	0.28	0.7 (4.4)	258 (284)	0.007	0.4 (0.4)	14.0 (15.3)
Summer	0.22	No Capacity	107 (101)	0.018	No Capacity[2]	11.8 (13.8)
Fall	0.20	No Capacity (2.8)	152 (243)	0.009	0.2 (0.3)	11.6 (14.2)

Notes:
1. 75th percentile of seasonal upstream concentrations.
2. No capacity due to elevated concentrations at the compliance point.
3. Values in brackets refer to predictions from the mass balance modelling approach, if different from the GoldSim modelling approach.
4. Unionized ammonia concentrations predicted in GoldSim based on modelled ammonia and average seasonal pH and temperature.
5. Unionized ammonia concentrations predicted using the mass balance approach based on measured concentrations and modelled as a conservative constituent.

According to Policy 2, during the summer, the effluent unionized ammonia concentration cannot exceed the upstream concentration of 0.018 mg/L. As such, the recommend effluent limits during the summer for unionized and total ammonia are 0.018 mg/L and 0.20 mg/L, respectively. Reliably achieving 0.20 mg/L total ammonia will be difficult for any nitrifying wastewater facility. Accordingly, 0.50 mg/L total ammonia concentration limits that are demonstrated in a nitrifying activated sludge system are recommended for summer conditions.

The predicted maximum allowable unionized ammonia concentrations listed in Table 28 for winter, spring, and fall exceed the acute toxicity guideline for unionized ammonia (0.10 mg/L as N). As such, it is recommended that the effluent limits for total ammonia be based on meeting the acute toxicity limit for unionized at end-of-pipe and 75th percentile water temperature and pH. Based on the resulting values presented in Table 29, the recommended total ammonia limit is recommended to be 1.4 mg/L for winter, spring, and fall. Accordingly, the recommended effluent limits for unionized and total ammonia in the summer are 0.50 mg/L and 1.4 mg/L, respectively.

Based on the information provided in Table 25, in terms of total ammonia discharge the required treatment level is equivalent to a membrane bioreactor at Location 1 is a membrane bioreactor with phosphorus removal and filtration.

Table 29: Maximum Allowable Seasonal Total Ammonia Concentrations for Discharge at Location 1 – Welland River East Based on Acute Toxicity Limits for Unionized Ammonia

Season	Ambient Conditions		Maximum Allowable Effluent Concentration (mg/L)	
	Water Temperature (ºC)	PH	Unionized Ammonia	Total Ammonia
Winter	2.1	7.82	0.1	15.2
Spring	14.4	8.23	0.1	2.36
Summer	25.3	8.26	0.018	**0.19**
Fall	20.5	8.27	0.1	**1.41**

Notes:
1. Lowest concentration reliably achievable in a nitrifying secondary treatment plant.

4.1.5 *E. coli*

The seasonal upstream geometric mean *E. coli* concentration ranges from 25 cfu/100 mL to 2,474 cfu/100 mL while the 75th percentile concentrations range from 105 cfu/100 mL to 6,920 cfu/100 mL. Since the upstream *E. coli* concentrations in the Welland River East consistently exceed the PWQO (100 cfu/100 mL), it is considered a Policy 2 receiver with respect to *E. coli*. As such, the effluent concentration is not to exceed background conditions. It is recommended that an effluent limit of 200 cfu/100 mL, consistent with other treatment plants in the area.

Table 30: Maximum Allowable Seasonal *E. coli* Concentrations for Discharge at Location 1 – Welland River East

Season	Upstream (mg/L)	Maximum Allowable Effluent Concentration (mg/L)	
		Local Compliance Point	System Compliance Point
Winter	6,920	No Capacity[2]	No Capacity
Spring	308		75,382 (78,132)
Summer	105		107,502 (88,800)
Fall	170		76,349 (69,113)

Notes:

1. 75th percentile of seasonal upstream concentrations.
2. No capacity due to elevated concentrations at the compliance point.
3. Values in brackets refer to predictions from the mass balance modelling approach, if different from the GoldSim modelling approach.

4.1.6 $CBOD_5$ and Dissolved Oxygen

The seasonal 25th percentile upstream dissolved oxygen concentrations range from 8.1 mg/L to 13.8 mg/L, which correspond to approximately levels in excess of 90% of the dissolved oxygen saturation concentration at the seasonal water temperatures. The upstream $CBOD_5$ values are typically less than 2 mg/L. This combination of conditions indicates that dissolved oxygen is not likely to restrict the discharge of oxygen consuming organic material.

The mass balance modelling suggests that the dissolved oxygen concentrations downstream of the discharge are not sensitive to the effluent dissolved oxygen concentrations.

The recommended annual maximum allowable $CBOD_5$ concentrations for effluent is based on the minimum value of 245 mg/L (fall) from Table 31. This value is well above the minimum secondary effluent standard limit of 25 mg/L (Table 25). As such, the recommended effluent limit for $CBOD_5$ is 25 mg/L. However, it should be noted that the treatment level required to achieve the phosphorus limits will result in an effluent $CBOD_5$ concentration of <5 mg/L.

Table 31: Maximum Allowable Seasonal $CBOD_5$ Concentrations for Discharge at Location 1 – Welland River East

Season	Upstream $CBOD_5$ (mg/L)[1]	Maximum Allowable Effluent Concentration (mg/L)[2]
Winter	1.3	371
Spring	1.0	384
Summer	2.0	**245**
Fall	1.0	289

Notes:

1. Upstream 75th percentile concentration.
2. Based on effluent dissolved oxygen concentration equal to 50% of saturation.

4.1.7 Total Suspended Solids (TSS)

The seasonal 75^{th} percentile upstream TSS concentrations range from 9.7 mg/L to 34.9 mg/L suggesting that the receiving water is not heavily impacted by suspended sediment. Based on the mass balance modelling results provided in Table 33, the recommended annual maximum allowable TSS concentration for effluent is 202 based on the minimum value (fall) from the table below.

This value is well above the minimum secondary effluent limit of 15 mg/L (Table 25). As such, the recommended effluent limit for TSS is 15 mg/L.

Table 32: Maximum Allowable Seasonal TSS Concentrations for Discharge at Location 1 – Welland River East

Season	Upstream TSS (mg/L)[1]	Maximum Allowable Effluent Concentration (mg/L)[2]
Winter	34.9	204
Spring	20.9	202
Summer	11.4	213
Fall	9.7	**202**

Notes:

1. Upstream 75^{th} percentile concentration.

4.1.8 Recommended Effluent Limits

Based on the preceding discussions, a summary of the recommended effluent limits for the Welland River East discharge is presented in Table 33.

Table 33: Summary of Development of Effluent Limits for Discharge at Location 1 – Welland River East

Parameter		Limiting Assimilative Capacity Concentration[1]	Typical Treatment Plant Effluent[2]	Proposed Effluent Limits
Total Phosphorus (mg/L)		0.10[3]	0.10	0.100
Nitrate (mg/L)		29	20	N/A[4]
Unionized Ammonia (mg/L)	Summer	0.018[3]	--	0.018
	Winter/Spring/Fall	0.1	--	0.10
Total Ammonia (mg/L)	Summer	0.2[3]	0.3	0.5
	Winter/Spring/Fall	1.4	0.3	1.4
E. coli (cfu/100 mL)		no capacity[3]	<100	200
Dissolved Oxygen (% of Saturation)		50%	>80%	N/A[4]
$CBOD_5$ (mg/L)		239	10	25
Total Suspended Solids (mg/L)		202	5	15

Notes:

1. lowest seasonal value from local and system compliance points.
2. typical effluent for a membrane bioreactor with phosphorus removal and filtration.
3. No capacity – Policy 2 receiver.
4. 4.Not applicable – typical effluent is expected to be better than the limiting assimilative capacity concentration.

4.2 Location 2 – Hydro Electric Power Canal (HECP)

4.2.1 Overview of Existing Conditions

The HEPC discharge would release effluent to the earth-cut section of the HECP between Triangle Island and the Montrose Gate (start of rock-cut section). The existing water in the HEPC is a combination of inflows from the Niagara River (Chippewa Creek), Lyons Creek, and Welland River East. Under normal conditions, the effluent is expected to travel downstream in the HEPC and eventually enter the Niagara River at the Sir Adam Beck GS. The local compliance point (A2) is in the HEPC just upstream of the Montrose Gate and the system compliance point (A5) is in the HECP below the existing Niagara Falls WWTP so that the combined effects of both plants are considered in the ACS. The HEPC discharge is not expected to affect water quality in Chippewa Creek, Welland River East, or in the Niagara River upstream of the Sir Adam Beck GS.

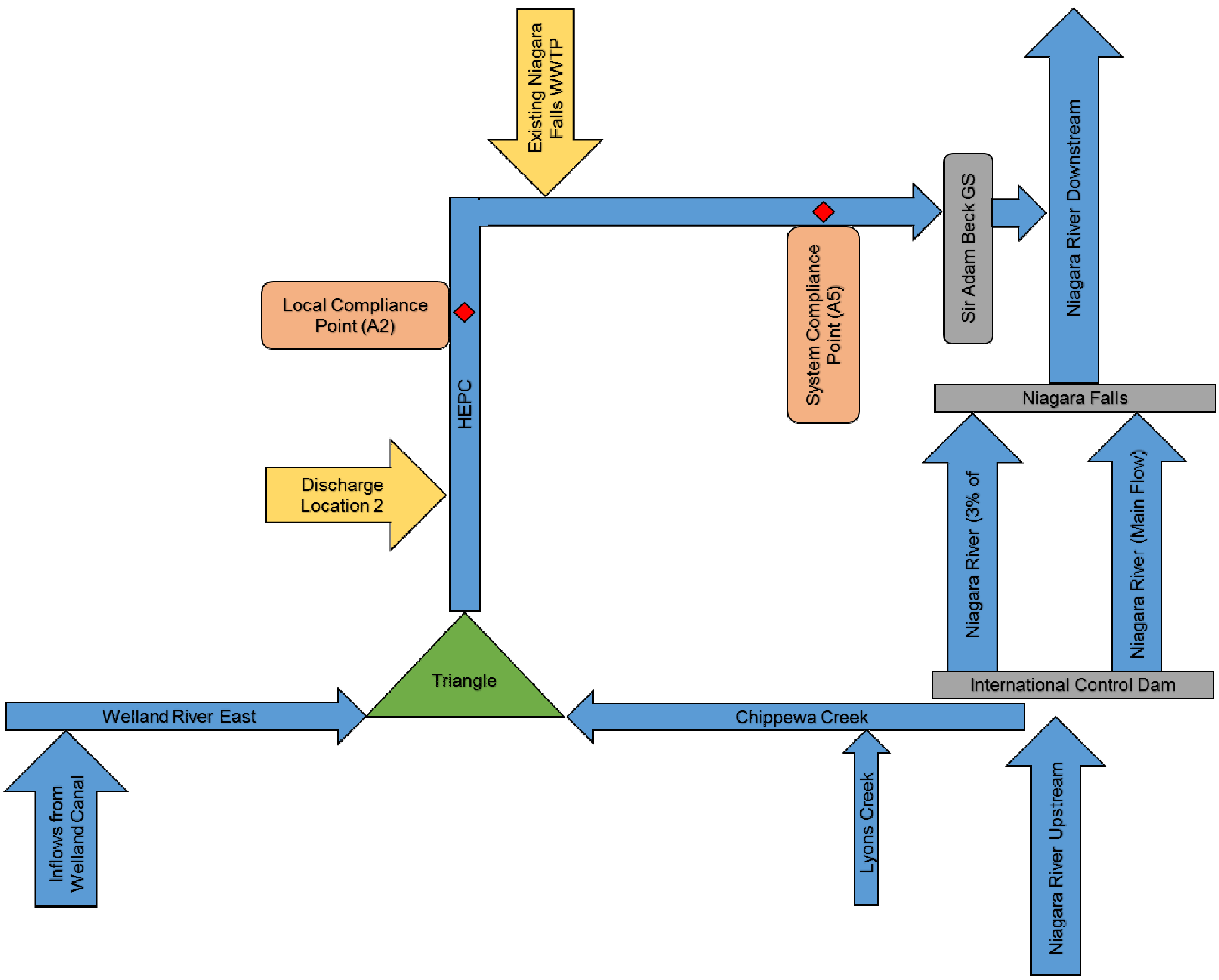


Figure 12: Local and System Compliance Points for Discharge at Location 2 – Hydro Electric Power Canal

4.2.2 Total Phosphorus

The total phosphorus concentrations in the HEPC are elevated in the winter, spring, and fall and consistently exceed the applicable PWQO (0.03 mg/L) in those seasons. The predicted seasonal 75th percentile concentrations range from 0.022 mg/L to 0.46 mg/L. Elevated total phosphorus concentrations in the HEPC are a result of elevated concentrations in the Niagara River during the winter and large phosphorus loads from Welland River East during the spring and fall. There are additional constraints at the system compliance point caused by the discharge of effluent into the HEPC from the existing Niagara Falls WWTP.

The predicted maximum allowable effluent concentrations for phosphorus are presented in Table 34. The elevated upstream total phosphorus concentrations result in Policy 2 conditions at the local and system compliance point in winter, spring, and fall. During summer, both the GoldSim and mass balance models show that effluent concentrations of 4.5 mg/L or more can be discharged to the HEPC without exceeding the total phosphorus target in the HEPC.

An effluent limit for total phosphorus of 0.75 mg/L is recommended based in the following rationale:

- The elevated phosphorus concentrations in the HEPC are the result of factors outside the study area (e.g., inflow from the Niagara River and Welland River East).
- The effluent flow rate represents less than 0.1% of the total flow in the HEPC and as such the contributions of the proposed discharge will cause negligible increases in the total phosphorus concentrations within the HEPC.
- Similarly, the effluent flow rate is insignificant when compared to the flow in the Niagara River below the Sir Adam beck GS.

Based on the information provided in Table 25, in terms of total phosphorus discharge the recommended treatment technology at Location 2 is a conventional activated sludge system with phosphorus removal and filtration.

Table 34: Maximum Allowable Seasonal Total Phosphorus Concentrations for Discharge at Location 2 – Hydro Electric Power Canal

Season	Upstream[1,2] (mg/L)	Maximum Allowable Effluent Concentration (mg/L)	
		Local Compliance Point	System Compliance Point
Winter	0.046 **(0.047)**	No Capacity	No Capacity
Spring	0.031 **(0.032)**	No Capacity	No Capacity
Summer	0.024 **(0.020)**	6.9 (5.7)	6.3 (5.0)
Fall	0.030 **(0.034)**	No Capacity	No Capacity

Notes:

1. Estimated value based on flow weighted average of inputs from Niagara River, Lyons Creek, and Welland River East at local compliance point during low flow conditions.
2. Values in bold indicate estimated value based on flow weighted average of inputs from Niagara River, Lyons Creek, and Welland River East at local compliance point during average conditions.
3. Values in brackets refer to predictions from the mass balance modelling approach, if different from the GoldSim modelling approach.

4.2.3 Nitrate

The predicted 75th percentile concentrations in the HEPC range from 0.18 mg/L to 0.37 mg/L. The highest nitrate concentrations typically occur during the winter. The predicted maximum allowable effluent concentrations for nitrate are presented in Table 35. In general, the local compliance point provides the most restrictive conditions. Based on the modelling results, the both the local and system compliance points can accept effluent nitrate concentrations in excess of 2,000 mg/L.

Based on the assumptions in Section 4.0, a conventional activated sludge system without denitrification is expected to provide effluent nitrate concentrations of 20 mg/L. As a result, nitrate limits would not be required for Location 2.

Table 35: Maximum Allowable Seasonal Nitrate Concentrations for Discharge at Location 2 – Hydro Electric Power Canal

Season	Upstream[1,2] (mg/L)	Maximum Allowable Effluent Concentration (mg/L)	
		Local Compliance Point	System Compliance Point
Winter	0.37 **0.31**	3,149 (2,644)	3,142 (2,629)
Spring	0.34 **0.31**	3,069 (2,681)	3,062(2,668)
Summer	0.27 **0.26**	3,334(2,750)	3,328 (2,740)
Fall	0.21 **0.18**	3,245(2,807)	3,238 (2,796)

Notes:

1. Estimated value based on flow weighted average of inputs from Niagara River, Lyons Creek, and Welland River East at local compliance point.
2. Values in bold indicate estimated value based on flow weighted average of inputs from Niagara River, Lyons Creek, and Welland River East at local compliance point during average conditions.
3. Values in brackets refer to predictions from the mass balance modelling approach, if different from the GoldSim modelling approach

4.2.4 Ammonia

The predicted 75th percentile concentrations for total ammonia in the HEPC range from 0.033 mg/L to 0.064 mg/L. The corresponding unionized ammonia concentrations are consistently below the applicable PWQO (0.0164 mg/L as N) for all the seasons. The maximum allowable effluent concentrations for total and unionized ammonia are presented in Table 36.

The predicted maximum allowable unionized ammonia concentrations listed in Table 36 exceed the acute toxicity guideline for unionized ammonia (0.10 mg/L as N). As such, it is recommended that the effluent limits for total ammonia be based on meeting the acute toxicity limit for unionized at end-of-pipe and seasonal water temperature and pH. The recommended effluent limit for unionized is 0.10 mg/L.

Based on the resulting values presented in Table 37, the recommended total ammonia limits are recommended to be 1.3 mg/L during the summer and 2.0 mg/L for the remainder if the year based on seasonal 75th percentile water temperature and pH in the HEPC.

Table 36: Maximum Allowable Seasonal Total and Unionized Ammonia Concentrations for Discharge at Location 2 – Hydro Electric Power Canal

Season	Total Ammonia			Unionized Ammonia		
	Upstream[1,2]	Maximum Allowable Concentration (mg/L)		Upstream[1,2]	Maximum Allowable Concentration (mg/L)	
		Local Compliance Point	System Compliance Point		Local Compliance Point	System Compliance Point
Winter	0.033 (**0.037**)	1.347 (1,227)	1,342 (1,216)	0.0011 **0.0010**	12.6 (15.5)	12.5 (15.5)
Spring	0.054 (**0.064**)	262 (284)	258 (275)	0.0013 **0.0012**	14.1 (15.3)	14.0 (15.3)
Summer	0.051 (**0.063**)	112 (113)	107 (101)	0.0028 **0.0014**	12.2 (13.9)	11.8 (13.8)
Fall	0.038 (**0.050**)	157 (254)	152 (243)	0.0024 **0.0012**	11.8 (14.2)	11.6 (14.2)

Notes:
1. Estimated value based on flow weighted average of inputs from Niagara River, Lyons Creek, and Welland River East at local compliance point.
2. Values in **bold** indicate estimated value based on flow weighted average of inputs from Niagara River, Lyons Creek, and Welland River East at local compliance point during average conditions.
3. Values in brackets refer to predictions from the mass balance modelling approach, if different from the GoldSim modelling approach.
4. Unionized ammonia concentrations predicted in GoldSim based on modelled ammonia and average seasonal pH and temperature.
5. Unionized ammonia concentrations predicted using the mass balance approach based on measured concentrations and modelled as a conservative constituent.

Table 37: Maximum Allowable Seasonal Total Ammonia Concentrations for Discharge at Location 2 – Hydro Electric Power Canal Based on Acute Toxicity Limits for Unionized Ammonia

Season	Ambient Conditions		Maximum Allowable Effluent Concentration (mg/L)	
	Water Temperature (ºC)	pH	Unionized Ammonia	Total Ammonia
Winter	3.5	7.99	0.1	9.39
Spring	18.6	8.16	0.1	**2.04**
Summer	23.6	8.22	0.1	**1.27**
Fall	13.5	8.14	0.1	3.08

4.2.5 *E. coli*

The predicted 75th percentile *E. coli* concentration in the HEPC ranges from 12 cfu/100 mL to 319 cfu/100 mL. The predicted *E. coli* concentration exceed the PWQO (100 cfu/100 mL) during the winter due to contributions from Welland River East at both the local and system compliance points. As such, the effluent concentration is not to exceed background conditions during the winter. As shown in Table 38, during the remaining seasons, there is capacity at both compliance points to accept effluent *E. coli* concentrations that exceed 60,000 cfu/100 mL. These allowable concentrations greatly exceed the expected effluent quality from a treatment plant.

It is recommended that an effluent limit of 200 cfu/100 mL be applied, consistent with other treatment plants in the area. With disinfection of the final effluent, any of the treatment plant can expect to meet these criteria.

Table 38: Maximum Allowable Seasonal *E. coli* Concentrations for Discharge at Location 2 – Hydro Electric Power Canal

Season	Upstream[1,2] (mg/L)	Maximum Allowable Effluent Concentration (mg/L)	
		Local Compliance Point	System Compliance Point
Winter	274 **319**	No Capacity	No Capacity
Spring	22 **36**	75,615 (78,132)	75,382 (78,132)
Summer	12 **13**	107,736 (88,800)	107,502 (88,800)
Fall	31 **34**	76,549 (69,113)	76,349 (69,113)

Notes:

1. Estimated value based on flow weighted average of inputs from Niagara River, Lyons Creek, and Welland River East at local compliance point.
2. Values in **bold** indicate estimated value based on flow weighted average of inputs from Niagara River, Lyons Creek, and Welland River East at local compliance point during average conditions.
3. Values in brackets refer to predictions from the mass balance modelling approach, if different from the GoldSim modelling approach

4.2.6 Biochemical Oxygen Demand ($CBOD_5$) and Dissolved Oxygen

The mass balance modelling suggests that the dissolved oxygen concentrations downstream of the discharge are not sensitive to the effluent dissolved oxygen concentrations.

The recommended annual maximum allowable $CBOD_5$ concentrations for effluent is based on the minimum value of 5,952 mg/L (fall) from Table 39. This value is well above the minimum secondary effluent standard limit of 25 mg/L (Table 25). As such, the recommended effluent limit for $CBOD_5$ is 25 mg/L.

Table 39: Maximum Allowable Seasonal $CBOD_5$ Concentrations for Discharge at Location 2 – Hydro Electric Power Canal

Season	Upstream $CBOD_5$ (mg/L)[1]	Maximum Allowable Effluent Concentration (mg/L)[2]
Winter	2.0	6,768
Spring		6,800
Summer		7,940
Fall		**5,952**

Notes:

1. Highest seasonal 75th percentile concentration in HEPC.
2. Based on effluent dissolved oxygen concentration equal to 50% of saturation.

4.2.7 Total Suspended Solids (TSS)

The annual 75th percentile upstream TSS is estimated to be 11.3 mg/L suggesting that the HEPC does not typically have high concentration od suspended solids. The mass balance modelling results provided in Table 40, the recommended annual maximum allowable TSS concentration for effluent is 5,046 based on the minimum value (summer and fall).

This value is well above the expected effluent from a conventional activated sludge system of 15 mg/L (Table 25). This value is well above the minimum secondary effluent standard limit of 25 mg/L (Table 25). As such, the recommended effluent limit TSS is 25 mg/L.

Table 40: Maximum Allowable Seasonal TSS Concentrations for Discharge at Location 2 – Hydro Electric Power Canal

<table>
<tr><th>Season</th><th>Upstream TSS (mg/L)[1]</th><th>Maximum Allowable Effluent Concentration (mg/L)[2]</th></tr>
<tr><td>Winter</td><td rowspan="4">11.3</td><td>5,047</td></tr>
<tr><td>Spring</td><td>5,047</td></tr>
<tr><td>Summer</td><td>5,046</td></tr>
<tr><td>Fall</td><td>5,046</td></tr>
</table>

Notes:

1. Annual 75th percentile concentration from Niagara River.

4.2.8 Recommended Effluent Limits

Based on the preceding discussions, a summary of the recommended effluent limits for the HEPC discharge is presented in Table 41.

Table 41: Summary of Development of Effluent Limits and Limits for Discharge at Location 2 – Hydro Electric Power Canal

<table>
<tr><th colspan="2">Parameter</th><th>Limiting Assimilative Capacity Concentration[1]</th><th>Typical Treatment Plant Effluent[2]</th><th>Proposed Effluent Limits</th></tr>
<tr><td colspan="2">Total Phosphorus (mg/L)</td><td>No capacity[3]</td><td>0.5</td><td>0.75</td></tr>
<tr><td colspan="2">Nitrate (mg/L)</td><td>2,620</td><td>20</td><td>N/A[4]</td></tr>
<tr><td colspan="2">Unionized Ammonia (mg/L)</td><td>0.1</td><td>--</td><td>0.1</td></tr>
<tr><td rowspan="2">Total Ammonia (mg/L)</td><td>Summer</td><td>1.3</td><td><1</td><td>1.3</td></tr>
<tr><td>Winter/Spring/Fall</td><td>2.0</td><td><3</td><td>2.0</td></tr>
<tr><td colspan="2">E. coli (cfu/100 mL)</td><td></td><td><100</td><td>200</td></tr>
<tr><td colspan="2">Dissolved Oxygen (% of Saturation)</td><td>50%</td><td>>80%</td><td>N/A[4]</td></tr>
<tr><td colspan="2">$CBOD_5$ (mg/L)</td><td>5,097</td><td>25</td><td>25</td></tr>
<tr><td colspan="2">Total Suspended Solids (mg/L)</td><td>5,046</td><td>25</td><td>25</td></tr>
</table>

Notes:

1. Lowest seasonal value from local and system compliance points.
2. Typical effluent for secondary effluent without filtration
3. No capacity – Policy 2 receiver.
4. 4.Not applicable – typical effluent is expected to be better than the limiting assimilative capacity concentration.

4.3 Location 3 – Chippewa Creek

4.3.1 Overview of Existing Conditions

The Chippewa Creek discharge would release effluent to the Chippewa Creek between Lyons Creek and Triangle Island. The existing water quality in Chippewa Creek is dominated by the water quality in the Niagara River. Under normal conditions, the effluent will travel downstream into the HEPC and eventually enter the Niagara River at the Sir Adam Beck GS. The local compliance point (A3) is in Chippewa Creek just upstream of Triangle Island and the system compliance point (A5) is in the HEPC below the existing Niagara Falls WWTP, so that the combined effects of both plants are considered in the ACS. The Chippewa Creek discharge is not expected to affect water quality in Welland River East or in the Niagara River upstream of the Sir Adam Beck GS.

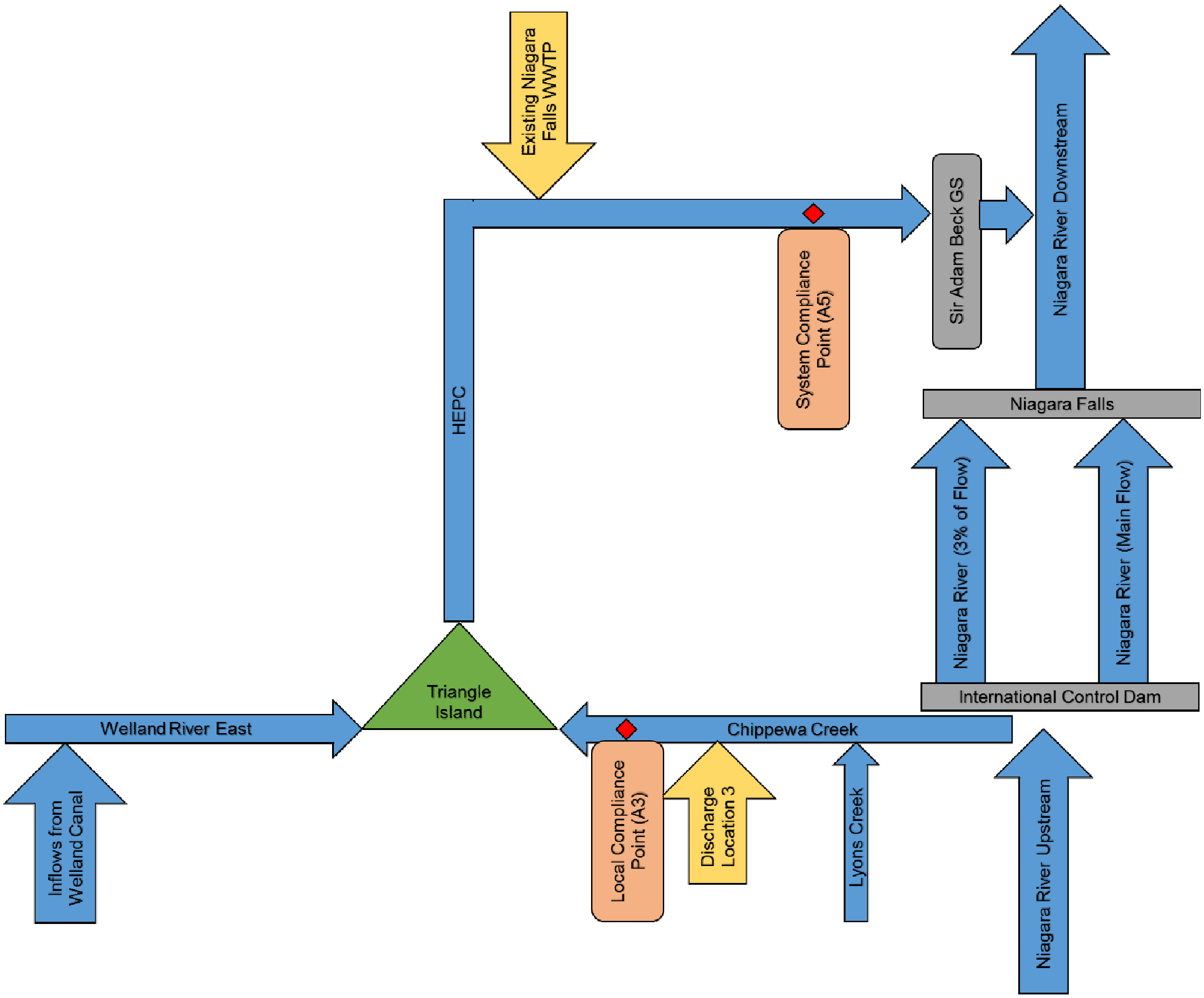


Figure 13: Local and System Compliance Points for Discharge at Location 3 – Chippewa Creek

4.3.2 Total Phosphorus

The measured seasonal 75th percentile concentrations of total phosphorus in Chippewa Creek range from 0.022 mg/L to 0.43 mg/L and are effectively the same as the measured conditions in the Niagara River. The total phosphorus concentrations in Chippewa are elevated in the winter as a result of elevated concentrations in the Niagara River during the winter. There are additional constraints at the system compliance point caused by the discharge of effluent into the HEPC from the existing Niagara Falls WWTP.

The predicted maximum allowable effluent concentrations for phosphorus are presented in Table 42. The elevated upstream total phosphorus concentrations result in Policy 2 conditions at the local compliance point in the winter months. At the local compliance point, Chippewa Creek can accept total phosphorus concentration of 2.8 mg/L or greater in the effluent in all the seasons except winter. At the system compliance point, elevated phosphorus concentrations are experienced in winter, spring and fall months due to inputs from the Welland River East and existing Niagara Falls WWTP.

An effluent limit for total phosphorus of 0.75 mg/L is recommended based in the following rationale:

- On an annual basis, there is sufficient capacity to accept an effluent concentration greater than 0.75 mg/L.
- The effluent flow rate represents less than 0.1% of the total flow in Chippewa Creek and as such the contributions of the proposed discharge will cause negligible increases in the total phosphorus concentrations within Chippewa Creek and the HEPC.
- The elevated phosphorus concentrations in Chippewa Creek are only experienced during the winter months, which is outside the algae growing season. The elevated winter background concentrations are the result of factors outside the study area (e.g., inflow from the Niagara River).
- Similarly, the effluent flow rate is insignificant when compared to the flow in the Niagara River below the Sir Adam beck GS.

Table 42: Maximum Allowable Seasonal Total Phosphorus Concentrations for Discharge at Location 3 – Chippewa Creek

Season	Upstream[1] (mg/L)	Maximum Allowable Effluent Concentration (mg/L)	
		Local Compliance Point	System Compliance Point
Winter	0.043	No Capacity	No Capacity
Spring	0.026	3.3 (3.8)	No Capacity
Summer	0.022	9.2 (7.7)	6.3 (5.0)
Fall	0.027	3.0 (2.8)	No Capacity

Notes:

1. Estimated value based on flow weighted average of inputs from Niagara River and Lyons Creek
2. Values in brackets refer to predictions from the mass balance modelling approach, if different from the GoldSim modelling approach

4.3.3 Nitrate

The measured 75th percentile nitrate concentrations in Chippewa Creek range from 0.18 mg/L to 0.31 mg/L. The highest nitrate concentrations typically occur during the winter. The predicted maximum allowable effluent concentrations for nitrate are presented in Table 43. In general, the local compliance point provides the most restrictive conditions. Based on the modelling results, the both the local and system compliance points can accept effluent nitrate concentrations in excess of 2,000 mg/L.

Based on the assumptions in Section 4.0, a conventional activated sludge system without denitrification is expected to provide effluent nitrate concentrations of 20 mg/L. As a result, nitrate limits would not be required for Location 3.

Table 43: Maximum Allowable Seasonal Nitrate Concentrations for Discharge at Location 3 – Chippewa Creek

Season	Upstream[1] (mg/L)	Maximum Allowable Effluent Concentration (mg/L)	
		Local Compliance Point	System Compliance Point
Winter	0.31	3,108 (2,621)	3,142 (2,629)
Spring	0.31	2,910 (2,614)	3,062 (2,668)
Summer	0.26	3,219 (2,652)	3,328 (2,740)
Fall	0.18	3,133 (2,735)	3,238 (2,796)

Notes:

1. Estimated value based on flow weighted average of inputs from Niagara River and Lyons Creek
2. Values in brackets refer to predictions from the mass balance modelling approach, if different from the GoldSim modelling approach

4.3.4 Ammonia

The measured 75th percentile concentrations for total ammonia in Chippewa Creek range from 0.014 mg/L to 0.032 mg/L. The corresponding unionized ammonia concentrations are consistently below the applicable PWQO (0.0164 mg/L as N) for all the seasons. The maximum allowable effluent concentrations for total and unionized ammonia are presented in Table 44.

The predicted maximum allowable unionized ammonia concentrations listed in Table 44: exceed the acute toxicity guideline for unionized ammonia (0.10 mg/L as N). As such, it is recommended that the effluent limits for total ammonia be based on meeting the acute toxicity limit for unionized at end-of-pipe and seasonal water temperature and pH.

Based on the resulting values presented in Table 45, the recommended total ammonia limits are recommended to be 1.0 mg/L during the summer and 1.7 mg/L for the remainder if the year based on seasonal average water temperature and pH in the HEPC.

Table 44: Maximum Allowable Seasonal Total and Unionized Ammonia Concentrations for Discharge at Location 3 – Chippewa Creek

Season	Total Ammonia			Unionized Ammonia		
	Upstream[1]	Maximum Allowable Concentration (mg/L)		Upstream[1]	Maximum Allowable Concentration (mg/L)	
		Local Compliance Point	System Compliance Point		Local Compliance Point	System Compliance Point
Winter	0.014	1,312 (1,294)	1,342 (1,216)	0.00012	12.12 (15.0)	12.52 (15.5)
Spring	0.046	261 (280)	258 (275)	0.00083	13.40 (15.0)	13.98 (15.3)
Summer	0.044	115 (115)	107 (101)	0.00339	12.24 (13.9)	11.82 (13.8)
Fall	0.032	159 (251)	152 (243)	0.00093	11.85 (14.0)	11.65 (14.2)

Notes:

1. Estimated value based on flow weighted average of inputs from Niagara River and Lyons Creek.
2. Values in brackets refer to predictions from the mass balance modelling approach, if different from the GoldSim modelling approach.
3. Unionized ammonia concentrations predicted in GoldSim based on modelled ammonia and average seasonal pH and temperature.
4. Unionized ammonia concentrations predicted using the mass balance approach based on measured concentrations and modelled as a conservative constituent.

Table 45: Maximum Allowable Seasonal Total Ammonia Concentrations for Discharge at Location 3 – Chippewa Creek Based on Acute Toxicity Limits for Unionized Ammonia

Season	Ambient Conditions		Maximum Allowable Effluent Concentration (mg/L)	
	Water Temperature (ºC)	pH	Unionized Ammonia	Total Ammonia
Winter	2.5	8.12	0.100	7.58
Spring	10.1	8.20	0.100	3.47
Summer	23.9	8.33	0.100	**0.99**
Fall	20.1	8.20	0.100	**1.68**

4.3.5 *E. coli*

The measured 75th percentile *E. coli* concentration in Chippewa Creek ranges from 8 cfu/100 mL to 50 cfu/100 mL and are consistently below the PWQO (100 cfu/100 mL). There are limitations on the discharge at the system compliance point during the winter due to contributions from Welland River East. As such, the effluent concentration is not to exceed background conditions during the winter. As shown in Table 46, during the remaining seasons, there is capacity at both compliance points to accept effluent *E. coli* concentrations that exceed 55,000 cfu/100 mL. These allowable concentrations greatly exceed the expected effluent quality from a treatment plant.

It is recommended that an effluent limit of 200 cfu/100 mL be used, consistent with other treatment plants in the area.

Table 46: Maximum Allowable Seasonal *E. coli* Concentrations for Discharge at Location 3 – Chippewa Creek

Season	Upstream[1] (mg/L)	Maximum Allowable Effluent Concentration (mg/L)	
		Local Compliance Point	System Compliance Point
Winter	50	55,235	No Capacity
Spring	12	94,761	75,382
Summer	8	107,502	107,502
Fall	26	81,586	76,349

Notes:

1. Estimated value based on flow weighted average of inputs from Niagara River and Lyons Creek
2. Values in brackets refer to predictions from the mass balance modelling approach, if different from the GoldSim modelling approach

4.3.6 Biochemical Oxygen Demand ($CBOD_5$) and Dissolved Oxygen

The mass balance modelling suggests that the dissolved oxygen concentrations downstream of the discharge are not sensitive to the effluent dissolved oxygen concentrations. As such, effluent dissolved oxygen concentrations equal to 50% of the saturation concentration are recommended as the effluent limit

The recommended annual maximum allowable $CBOD_5$ concentrations for effluent is based on the minimum value of 5,707 mg/L (fall) from Table 47. This value is well above the minimum secondary effluent standard limit of 25 mg/L (Table 25). As such, the recommended effluent limit for $CBOD_5$ is 25 mg/L.

Table 47: Maximum Allowable Seasonal $CBOD_5$ Concentrations for Discharge at Location 3 – Chippewa Creek

Season	Upstream $CBOD_5$ (mg/L)[1]	Maximum Allowable Effluent Concentration (mg/L)[2]
Winter	2.0	6,380
Spring		6,384
Summer		7,689
Fall		**5,707**

Notes:

1. Highest seasonal 75th percentile concentration in Welland River East.
2. Based on effluent dissolved oxygen concentration equal to 50% of saturation.

4.3.7 Total Suspended Solids (TSS)

The annual 75^{th} percentile upstream TSS is estimated to be 11.3 mg/L suggesting that Chippewa Creek does not typically have high concentration od suspended solids. The mass balance modelling results provided in Table 48, the recommended annual maximum allowable TSS concentration for effluent is 4,846 based on the minimum value (summer and fall). This value is well above the minimum secondary effluent standard limit of 25 mg/L (Table 25). As such, the recommended effluent limit TSS is 25 mg/L.

Table 48: Maximum Allowable Seasonal TSS Concentrations for Discharge at Location 3 – Chippewa Creek

Season	Upstream TSS (mg/L)[1]	Maximum Allowable Effluent Concentration (mg/L)[2]
Winter	11.3	4,880
Spring		4,866
Summer		**4,846**
Fall		4,855

Notes:

2. Annual 75^{th} percentile concentration from Niagara River.

4.3.8 Recommended Effluent Limits

Based on the preceding discussions, a summary of the recommended effluent concentrations for the Chippewa Creek discharge is presented in Table 49. In order to meet the limits and limits for each parameter, if the new WWTP discharges to Chippewa Creek the new plant would be designed as a membrane bioreactor with phosphorus removal and filtration. This advanced level of treatment is required in order to meet the end-of-pipe acute toxicity criteria during the summer.

Table 49: Summary of Development of Effluent Limits for Discharge at Location 3 – Chippewa Creek

Parameter		Limiting Assimilative Capacity Concentration[1]	Typical Treatment Plant Effluent[2]	Proposed Effluent Limits
Total Phosphorus (mg/L)		No capacity[3]	0.5	0.75
Nitrate (mg/L)		2,614	20	N/A[4]
Unionized Ammonia (mg/L)		0.1	--	0.10
Total Ammonia (mg/L)	Summer	1.0	<1	1.1
	Winter/Spring/Fall	1.7	<3	1.7
E. coli (cfu/100 mL)		55,235	100	200
Dissolved Oxygen (% of Saturation)		50%	>80%	N/A[4]
$CBOD_5$ (mg/L)		4,885	25	25
Total Suspended Solids (mg/L)		4,846	25	25

Notes:

1. Lowest seasonal value from local and system compliance points.
2. Typical effluent for a conventional activated sludge without filtration.
3. No capacity – Policy 2 receiver during winter months only.
4. Not applicable – typical effluent is expected to be better than the limiting assimilative capacity concentration.

4.4 Location 4 – Niagara River

4.4.1 Overview of Existing Conditions

The Niagara River discharge would release effluent to the Niagara River just downstream of the ICD approximately 1.8 km upstream of Niagara Falls. The effluent is expected to form a shoreline plume as it travels downstream to the falls. The effluent is expected to mix with approximately 3% of the total flow in the Niagara River in the 10-minute travel time. Below the falls, the effluent is expected to mix completely with the Niagara River flow. The local compliance point (A4) is located on the Canadian shoreline at the crest of the falls. There is no system compliance point for this location since the Niagara River discharge not expected to affect water quality in Welland River East, Chippewa Creek, in the HEPC where the existing Niagara Falls WWTP discharges into. There is no system compliance point for this location since the Niagara River discharge not expected to affect water quality in Welland River East, Chippewa Creek, in the HEPC where the existing Niagara Falls WWTP discharges into.

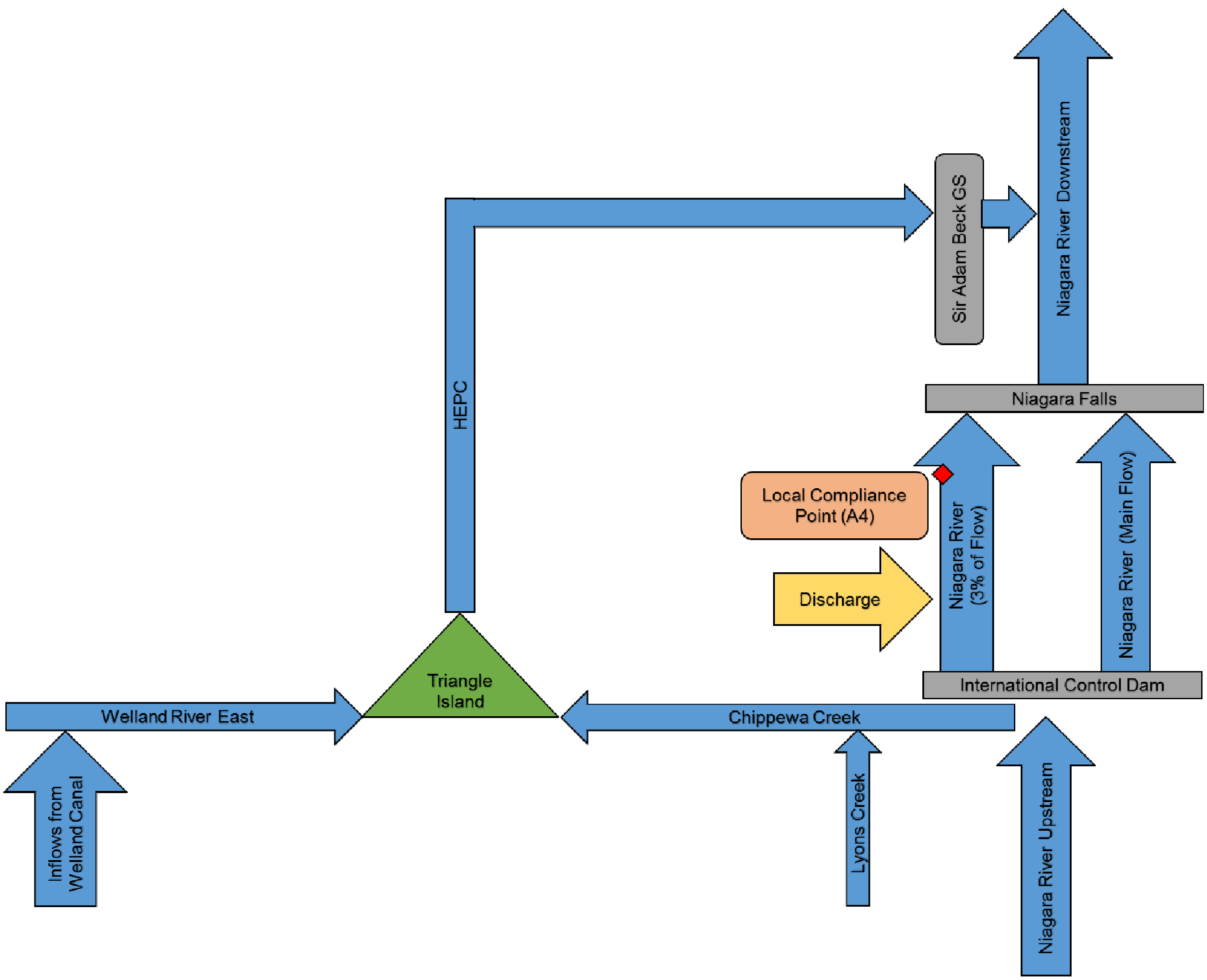


Figure 14: Local and System Compliance Points for Discharge at Location 4 – Niagara River

4.4.2 Total Phosphorus

The measured seasonal 75th percentile concentrations of total phosphorus in Niagara River range from 0.022 mg/L to 0.43 mg/L. The total phosphorus concentrations in Niagara River are elevated in the winter and result in discharge constraints in the winter.

The predicted maximum allowable effluent concentrations for phosphorus are presented in Table 50. The elevated upstream total phosphorus concentrations result in Policy 2 conditions at the local compliance during winter months. At the local compliance point, the Niagara River can accept total phosphorus concentration of 0.58 mg/L or greater in the effluent in all the seasons except winter.

An effluent limit for total phosphorus of 0.75 mg/L is recommended based in the following rationale:

- On an annual basis, there is sufficient capacity to accept an effluent concentration greater than 0.75 mg/L.
- The elevated phosphorus concentrations in the Niagara River are only during winter months and are the result of factors outside the study area (e.g., upstream in the Niagara River and Lake Erie).
- The effluent flow rate represents less than 0.01% of the total flow in Niagara River and as such the contributions of the proposed discharge will cause negligible increases in the total phosphorus concentrations downstream.

Table 50: Maximum Allowable Seasonal Total Phosphorus Concentrations for Discharge at Location 4 – Niagara River

Season	Upstream (mg/L)	Maximum Allowable Effluent Concentration (mg/L)
Winter	0.043	No Capacity
Spring	0.026	0.764
Summer	0.022	1.498
Fall	0.027	0.581

4.4.3 Nitrate

The measured 75th percentile nitrate concentrations in the Niagara River range from 0.18 mg/L to 0.31 mg/L. The highest nitrate concentrations typically occur during the winter. The predicted maximum allowable effluent concentrations for nitrate are presented in Table 53:. Based on the modelling results, the Niagara River can accept effluent nitrate concentrations in of 497 mg/L or greater.

Based on the assumptions in Section 4.0, a conventional activated sludge system without denitrification is expected to provide effluent nitrate concentrations of 20 mg/L. As a result, nitrate limits would not be required for Location 3.

Table 51: Maximum Allowable Seasonal Nitrate Concentrations for Discharge at Location 4 – Niagara River

Season	Upstream (mg/L)	Maximum Allowable Effluent Concentration (mg/L)
Winter	0.31	**497**
Spring	0.31	497
Summer	0.26	577
Fall	0.18	521

4.4.4 Ammonia

The measured 75th percentile concentrations for total ammonia in Niagara River range from 0.014 mg/L to 0.032 mg/L. The corresponding unionized ammonia concentrations are consistently below the applicable PWQO (0.0164 mg/L as N) for all the seasons. The maximum allowable effluent concentrations for total and unionized ammonia are presented in Table 52.

The predicted maximum allowable unionized ammonia concentrations listed in Table 52 exceed the acute toxicity guideline for unionized ammonia (0.10 mg/L as N). As such, it is recommended that the effluent limit for total ammonia be based on meeting the acute toxicity limit for unionized at end-of-pipe and seasonal water temperature and pH.

Based on the resulting values presented in Table 53:, the recommended total ammonia limits are recommended to be 1.0 mg/L during the summer and 1.7 mg/L for the remainder if the year based on seasonal average water temperature and pH in the HEPC.

Table 52: Maximum Allowable Seasonal Total and Unionized Ammonia Concentrations for Discharge at Location 4 – Niagara River

Season	Upstream (mg/L))		Maximum Allowable Effluent Concentration (mg/L)	
	Total	Unionized	Total	Unionized
Winter	0.014	0.00012	227	3.0
Spring	0.046	0.00083	97	2.8
Summer	0.044	0.00339	25	2.5
Fall	0.032	0.00093	45	2.7

Table 53: Maximum Allowable Seasonal Total Ammonia Concentrations for Discharge at Location 4 – Niagara River Based on Acute Toxicity Limits for Unionized Ammonia

Season	Ambient Conditions		Maximum Allowable Effluent Concentration (mg/L)	
	Water Temperature (°C)	pH	Unionized Ammonia	Total Ammonia
Winter	2.5	8.12	0.100	7.58
Spring	10.1	8.20	0.100	3.47
Summer	23.9	8.33	0.100	**0.99**
Fall	20.1	8.20	0.100	**1.68**

4.4.5 *E. coli*

The measured 75th percentile *E. coli* concentration in the Niagara River ranges from 8 cfu/100 mL to 50 cfu/100 mL and are consistently below the PWQO (100 cfu/100 mL). There are no seasonal limitations on the discharge identified. As shown in Table 54, there is capacity in all seasons to accept effluent *E. coli* concentrations that exceed 9,000 cfu/100 mL. These allowable concentrations greatly exceed the expected effluent quality from a treatment plant.

It is recommended that an effluent limit of 200 cfu/100 mL be used, consistent with other treatment plants in the area.

Table 54: Maximum Allowable Seasonal Nitrate Concentrations for Discharge at Location 4 – Niagara River

Season	Upstream (cfu/100 mL)	Maximum Allowable Effluent Concentration (cfu/100 mL)
Winter	50	**9,276**
Spring	12	16,249
Summer	8	19,368
Fall	26	13,680

4.4.6 Biochemical Oxygen Demand ($CBOD_5$) and Dissolved Oxygen

The mass balance modelling suggests that the dissolved oxygen concentrations downstream of the discharge are not sensitive to the effluent dissolved oxygen concentrations.

The recommended annual maximum allowable $CBOD_5$ concentrations for effluent is based on the minimum value of 1,083 mg/L (fall) from Table 55. This value is well above the minimum secondary effluent standard limit of 25 mg/L (Table 25). As such, the recommended effluent limit for $CBOD_5$ is 25 mg/L.

Table 55: Maximum Allowable Seasonal $CBOD_5$ Concentrations for Discharge at Location 4 – Niagara River

Season	Upstream $CBOD_5$ (mg/L)[1]	Maximum Allowable Effluent Concentration (mg/L)[2]
Winter	2.0	1,204
Spring		1,275
Summer		1,461
Fall		**1,083**

Notes:

1. Highest seasonal 75th percentile concentration in Welland River East.
2. Based on effluent dissolved oxygen concentration equal to 50% of saturation.

4.4.7 Total Suspended Solids (TSS)

The annual 75th percentile upstream TSS is estimated to be 11.3 mg/L suggesting that the Niagara River does not typically have high concentration od suspended solids. The mass balance modelling results provided in

Table 56, the recommended annual maximum allowable TSS concentration for effluent is 934 based on the minimum value. This value is well above the minimum secondary effluent standard limit of 25 mg/L (Table 25). As such, the recommended effluent limit TSS is 25 mg/L.

Table 56: Maximum Allowable Seasonal TSS Concentrations for Discharge at Location 4 – Niagara River

Season	Upstream TSS (mg/L)[1]	Maximum Allowable Effluent Concentration (mg/L)[2]
Winter	11.3	**934**
Spring		985
Summer		**934**
Fall		**934**

Notes:

1. Annual 75th percentile concentration from Niagara River.

4.4.8 Recommended Effluent Limits

Based on the preceding discussions, a summary of the recommended effluent concentrations for the Niagara River discharge is presented in Table 57.

Table 57: Summary of Development of Effluent Limits for Discharge at Location 4 – Niagara River

Parameter		Limiting Assimilative Capacity Concentration[1]	Typical Treatment Plant Effluent[2]	Proposed Effluent Limits
Total Phosphorus (mg/L)		No capacity[3]	0.5	0.5
Nitrate (mg/L)		497	20	N/A[4]
Unionized Ammonia (mg/L)		0.10	0.1	0.1
Total Ammonia (mg/L)	Summer	1.0	<1	1.0
	Winter/Spring/Fall	1.7	<3	1.7
E. coli (cfu/100 mL)		9,276	<100	200
Dissolved Oxygen (% of Saturation)		50%	>80%	N/A[4]
$CBOD_5$ (mg/L)		927	25	25
Total Suspended Solids (mg/L)		934	25	25

Notes:

1. Lowest seasonal value.
2. Typical effluent for a conventional activated sludge without filtration.
3. No capacity – Policy 2 receiver during winter months only.
4. 4.Not applicable – typical effluent is expected to be better than the limiting assimilative capacity concentration.

5.0 CUMULATIVE EFFECTS OF THE PROJECT ON WATER QUALITY

The following subsections of this report present the projected cumulative effect of different discharge location alternatives on receiving water quality within the system at downstream assessment points with accompanying discussion of seasonal sensitivities, where relevant. It should be noted that presented results specifically consider the effects of the proposed effluent discharge under the 7Q20 flow and 75th percentile condition, meaning that water quality conditions would typically be better than presented. A schematic of the mass balance model including the assessment points used in the cumulative effects assessment is provided in Figure 10.

5.1 Total Phosphorus

Table 58 compares the water quality effects of proposed discharge location alternatives at each of six assessment points recognising that the phosphorus effluent limit for discharge location 1 is limited to 0.1 mg/L due to Policy 2 conditions while the phosphorus effluent limit for discharge locations 2, 3 and 4 is 0.75 mg/L which are achievable in conventional activated sludge system with phosphorus removal.

As observed in the tables below, the WWTP at discharge location 1 results in the smallest cumulative change in downstream phosphorus concentrations. Total phosphorus concentrations at Assessment Point A1 generally decrease due to the intensified level of treatment and poor background water quality in Welland River East. Marginal increases in phosphorus concentrations are observed further downstream at Assessment Point A2 during the winter and fall and, on average, over the course of the year.

Owing to the higher phosphorus effluent limit at discharge locations 2, 3 and 4, the effect of the new WWTP at each of these locations at downstream assessment points (A2, A5 and A6 for discharge location 2; A3, A2, A5 and A6 for discharge location 3; A4 and A6 for discharge location 4) is slightly higher than for discharge location 1. However, that these increases are typically less than 0.1 µg/L (approximately 1.5%) and do not result in exceedances of the PWQO for phosphorus during the summer when the risk of algal growth is elevated.

To further demonstrate the effect of the Project on the total phosphorus concentrations, GoldSim was used to predict the expected distribution of total phosphorus concentrations at each of the assessment locations. This was accomplished completing a Monte Carlo simulation for each season and discharge location using statistical distributions of inflows (same as used in to estimate maximum allowable effluent concentrations) and statistical distributions of the total phosphorus concentration in the Niagara River, Lyons Creek, and Welland River East. In all cases, a log-normal distribution was used.

The results of this analysis are provided in Appendix A. For the discharge options into the HEPC and Chippewa Creek, the predicted distributions at all the affected assessment points are nearly identical to the baseline condition. For the discharge option to Welland River East, there is a predicted change to the distribution at Assessment Point A1 (a shift of the distribution to the right) suggesting an increase in total phosphorus concentrations.

Based on these two assessments, it is expected that the change in phosphorus concentrations in the receiving waters as a result of the Project will not be measurable for all cases except for the discharge into the Welland River East.

Table 58: Predicted Total Phosphorus Concentrations at Assessment Points by Season and Discharge Location

	Winter	Spring	Summer	Fall	Annual
New WWTP Flow (m^3/s)	0.35				
New WWTP Total Phosphorus Limit	0.1 mg/L at L1; 0.75 mg/L at L2, L3, L4				
A1 – Welland River East at Triangle Island					
Existing Concentration (µg/L) – No Discharge	140.0	160.0	80.0	100.0	118.2
Future Concentration (µg/L) – Discharge at L1	138.8	158.3	80.5	100.0	117.7
Future Concentration (µg/L) – Discharge at L2	140.0	160.0	80.0	100.0	118.2
Future Concentration (µg/L) – Discharge at L3	140.0	160.0	80.0	100.0	118.2
Future Concentration (µg/L) – Discharge at L4	140.0	160.0	80.0	100.0	118.2
A2 – HEPC at Montrose Gate					
Existing Concentration (µg/L) – No Discharge	46.2	30.8	24.4	29.8	32.7
Future Concentration (µg/L) – Discharge at L1	46.3	30.9	24.5	29.9	32.8
Future Concentration (µg/L) – Discharge at L2	46.9	31.5	25.1	30.5	33.5
Future Concentration (µg/L) – Discharge at L3	46.9	31.5	25.1	30.5	33.5
Future Concentration (µg/L) – Discharge at L4	46.2	30.8	24.4	29.8	32.7
A3 – Chippewa Creek at Triangle Island					
Existing Concentration (µg/L) – No Discharge	43.1	26.1	22.1	27.1	29.6
Future Concentration (µg/L) – Discharge at L1	43.1	26.1	22.1	27.1	29.6
Future Concentration (µg/L) – Discharge at L2	43.1	26.1	22.1	27.1	29.6
Future Concentration (µg/L) – Discharge at L3	43.8	26.9	22.9	27.8	30.3
Future Concentration (µg/L) – Discharge at L4	43.1	26.1	22.1	27.1	29.6
A4 – Niagara River at Falls (Canadian Shore)					
Existing Concentration (µg/L) – No Discharge	43.0	26.0	22.0	27.0	29.4
Future Concentration (µg/L) – Discharge at L1	43.0	26.0	22.0	27.0	29.4
Future Concentration (µg/L) – Discharge at L2	43.0	26.0	22.0	27.0	29.4
Future Concentration (µg/L) – Discharge at L3	43.0	26.0	22.0	27.0	29.4
Future Concentration (µg/L) – Discharge at L4	47.1	29.9	26.1	31.1	33.4
A5 – HEPC at Sir Adam Beck GS					
Existing Concentration (µg/L) – No Discharge	47.8	32.4	26.0	31.4	34.4
Future Concentration (µg/L) – Discharge at L1	47.9	32.5	26.1	31.5	34.4
Future Concentration (µg/L) – Discharge at L2	48.5	33.1	26.7	32.1	35.1
Future Concentration (µg/L) – Discharge at L3	48.5	33.1	26.7	32.1	35.1
Future Concentration (µg/L) – Discharge at L4	47.8	32.4	26.0	31.4	34.4
A6 – Niagara River below Sir Adam Beck GS					
Existing Concentration (µg/L) – No Discharge	43.6	26.8	22.5	27.5	30.0
Future Concentration (µg/L) – Discharge at L1	43.6	26.8	22.5	27.6	30.0
Future Concentration (µg/L) – Discharge at L2	43.7	26.9	22.6	27.6	30.1
Future Concentration (µg/L) – Discharge at L3	43.7	26.9	22.6	27.6	30.1
Future Concentration (µg/L) – Discharge at L4	43.7	26.9	22.6	27.6	30.1

5.2 Unionized Ammonia

Table 59 compares the water quality effects of proposed discharge location alternatives at each of six assessment points recognising that the unionized ammonia effluent limit for discharge location 1 is limited to 0.018 mg/L during the summer (membrane bioreactor with phosphorus removal and filtration) because existing background water quality in this watercourse is close to the PWQO of 0.0164 mg/L as N. The unionized ammonia effluent limit that has been applied during all other seasons and at all other discharge locations is 0.1 mg/L.

The effect of introducing the new WWTP at discharge locations 1 and 4 on local assessment points is conspicuous when compared to siting the new WWTP at discharge locations 2 and 3. Only minor differences in water quality effects between the four discharge locations are in evidence by the time the mixed effluent stream reaches the system assessment point (A5) and final assessment point (A6) indicating that water quality effects for unionized ammonia are relatively localized.

Table 59: Predicted Unionized Ammonia Concentrations at Assessment Points by Season and Discharge Location

	Winter	Spring	Summer	Fall	Annual
New WWTP Flow (m^3/s)	0.35				
New WWTP Unionized Ammonia Limit	18 µg/L at L1 (summer); otherwise 100 µg/L				
A1 – Welland River East at Triangle Island					
Existing Concentration (µg/L) – No Discharge	1.00	6.00	18.00	9.00	8.94
Future Concentration (µg/L) – Discharge at L1	3.93	8.59	18.00	11.37	10.83
Future Concentration (µg/L) – Discharge at L2	1.00	6.00	18.00	9.00	8.94
Future Concentration (µg/L) – Discharge at L3	1.00	6.00	18.00	9.00	8.94
Future Concentration (µg/L) – Discharge at L4	1.00	6.00	18.00	9.00	8.94
A2 – HEPC at Montrose Gate					
Existing Concentration (µg/L) – No Discharge	1.00	1.18	2.63	2.26	1.77
Future Concentration (µg/L) – Discharge at L1	1.10	1.28	2.64	2.36	1.85
Future Concentration (µg/L) – Discharge at L2	1.10	1.28	2.72	2.36	1.87
Future Concentration (µg/L) – Discharge at L3	1.10	1.28	2.72	2.36	1.87
Future Concentration (µg/L) – Discharge at L4	1.00	1.18	2.63	2.26	1.77
A3 – Chippewa Creek at Triangle Island					
Existing Concentration (µg/L) – No Discharge	1.00	1.00	2.00	2.01	1.50
Future Concentration (µg/L) – Discharge at L1	1.00	1.00	2.00	2.01	1.50
Future Concentration (µg/L) – Discharge at L2	1.00	1.00	2.00	2.01	1.50
Future Concentration (µg/L) – Discharge at L3	1.10	1.11	2.10	2.11	1.60
Future Concentration (µg/L) – Discharge at L4	1.00	1.00	2.00	2.01	1.50
A4 – Niagara River at Falls (Canadian Shore)					
Existing Concentration (µg/L) – No Discharge	1.00	1.00	2.00	2.00	1.49
Future Concentration (µg/L) – Discharge at L1	1.00	1.00	2.00	2.00	1.49
Future Concentration (µg/L) – Discharge at L2	1.00	1.00	2.00	2.00	1.49
Future Concentration (µg/L) – Discharge at L3	1.00	1.00	2.00	2.00	1.49
Future Concentration (µg/L) – Discharge at L4	1.54	1.52	2.54	2.54	2.03
A5 – HEPC as Sir Adam Beck GS					
Existing Concentration (µg/L) – No Discharge	1.07	1.25	2.75	2.36	1.86
Future Concentration (µg/L) – Discharge at L1	1.17	1.35	2.77	2.46	1.94
Future Concentration (µg/L) – Discharge at L2	1.17	1.35	2.85	2.46	1.96
Future Concentration (µg/L) – Discharge at L3	1.17	1.35	2.85	2.46	1.96
Future Concentration (µg/L) – Discharge at L4	1.07	1.25	2.75	2.36	1.86
A6 – Niagara River below Sir Adam Beck GS					
Existing Concentration (µg/L) – No Discharge	1.01	1.03	2.09	2.05	1.54
Future Concentration (µg/L) – Discharge at L1	1.02	1.04	2.10	2.06	1.55
Future Concentration (µg/L) – Discharge at L2	1.02	1.04	2.11	2.06	1.55
Future Concentration (µg/L) – Discharge at L3	1.02	1.04	2.11	2.06	1.55
Future Concentration (µg/L) – Discharge at L4	1.02	1.04	2.11	2.06	1.55

5.3 Total Ammonia

Table 60 compares the water quality effects of proposed discharge location alternatives at each of six assessment points for total ammonia. In each case the total ammonia was estimated using the unionized ammonia effluent limits (discussed in Section 5.2), the average seasonal water temperature and pH within each receiver. The below water quality results for total ammonia thus reflect a variety of seasonal and location-based water quality and temperature characteristics.

The tabulated results indicate that water quality at local assessment points, particularly at A1, can be substantially influenced by introducing the new WWTP upstream. As would be expected, the magnitude of these influences decreases considerably with distance downstream as the influence of other loadings sources and flows becomes more dominant.

As no provincial water quality limit is tied directly to total ammonia, the significance of water quality effects of discharge location alternatives at each assessment is best evaluated for unionized ammonia (Section 5.2).

Table 60: Predicted Total Ammonia Concentrations at Assessment Points by Season and Discharge Location

	Winter	Spring	Summer	Fall	Annual
New WWTP Flow (m^3/s)	0.35				
New WWTP Total Ammonia Limit	1.4 mg/L (winter, spring, fall) & 0.5 mg/L (summer) at L1; 1.3 mg/L (winter, spring, fall) & 2.0 mg/L (summer) at L2; 1.0 mg/L (winter, spring, fall) & 1.7 mg/L (summer) at L3 & L4				
A1 - Welland River East at Triangle Island					
Existing Concentration - No Discharge (mg/L)	0.2300	0.2100	0.2200	0.2000	0.2146
Future Concentration - Discharge at L1 (mg/L)	0.2646	0.2428	0.2270	0.2313	0.2404
Future Concentration - Discharge at L2 (mg/L)	0.2300	0.2100	0.2200	0.2000	0.2146
Future Concentration - Discharge at L3 (mg/L)	0.2300	0.2100	0.2200	0.2000	0.2146
Future Concentration - Discharge at L4 (mg/L)	0.2300	0.2100	0.2200	0.2000	0.2146
A2 - HEPC at Montrose Gate					
Existing Concentration - No Discharge (mg/L)	0.0240	0.0518	0.0509	0.0238	0.0377
Future Concentration - Discharge at L1 (mg/L)	0.0253	0.0531	0.0513	0.0252	0.0389
Future Concentration - Discharge at L2 (mg/L)	0.0253	0.0531	0.0511	0.0252	0.0388
Future Concentration - Discharge at L3 (mg/L)	0.0256	0.0534	0.0518	0.0255	0.0392
Future Concentration - Discharge at L4 (mg/L)	0.0240	0.0518	0.0509	0.0238	0.0377
A3 - Chippewa Creek at Triangle Island					
Existing Concentration - No Discharge (mg/L)	0.0170	0.0461	0.0440	0.0170	0.0311
Future Concentration - Discharge at L1 (mg/L)	0.0170	0.0461	0.0440	0.0170	0.0311
Future Concentration - Discharge at L2 (mg/L)	0.0170	0.0461	0.0440	0.0170	0.0311
Future Concentration - Discharge at L3 (mg/L)	0.0187	0.0478	0.0450	0.0188	0.0327
Future Concentration - Discharge at L4 (mg/L)	0.0170	0.0461	0.0440	0.0170	0.0311
A4 - Niagara River at Falls (Canadian Shore)					
Existing Concentration - No Discharge (mg/L)	0.0170	0.0460	0.0440	0.0170	0.0313
Future Concentration - Discharge at L1 (mg/L)	0.0170	0.0460	0.0440	0.0170	0.0313
Future Concentration - Discharge at L2 (mg/L)	0.0170	0.0460	0.0440	0.0170	0.0313
Future Concentration - Discharge at L3 (mg/L)	0.0170	0.0460	0.0440	0.0170	0.0313
Future Concentration - Discharge at L4 (mg/L)	0.0263	0.0548	0.0494	0.0263	0.0395
A5 - HEPC at Sir Adam Beck GS					
Existing Concentration - No Discharge (mg/L)	0.0456	0.0683	0.0698	0.0419	0.0565
Future Concentration - Discharge at L1 (mg/L)	0.0470	0.0696	0.0702	0.0432	0.0576
Future Concentration - Discharge at L2 (mg/L)	0.0470	0.0696	0.0699	0.0432	0.0575
Future Concentration - Discharge at L3 (mg/L)	0.0472	0.0699	0.0707	0.0435	0.0580
Future Concentration - Discharge at L4 (mg/L)	0.0456	0.0683	0.0698	0.0419	0.0565
A6 - Niagara River Below Sir Adam Beck					
Existing Concentration - No Discharge (mg/L)	0.0205	0.0487	0.0472	0.0201	0.0344
Future Concentration - Discharge at L1 (mg/L)	0.0207	0.0488	0.0473	0.0203	0.0345
Future Concentration - Discharge at L2 (mg/L)	0.0207	0.0488	0.0472	0.0203	0.0345
Future Concentration - Discharge at L3 (mg/L)	0.0208	0.0489	0.0473	0.0203	0.0346
Future Concentration - Discharge at L4 (mg/L)	0.0208	0.0489	0.0473	0.0203	0.0346

5.4 Nitrate

Table 61 compares the water quality effects of proposed discharge location alternatives at each of six assessment points with the conventional secondary treatment effluent nitrate concentrations of 20 mg/L being applied consistently across seasons and locations. This concentration is consistent with a fully nitrifying facility without denitrification.

Notable from the results is that the new WWTP has a negligible effect on nitrate concentrations within receiving waters in all cases except at assessment point A1 when discharge location 1 is considered. In this case increases in nitrate concentrations of between 25% and 100% are observed, depending on season. Even so, these changes are not considered significant from a water quality perspective because instream nitrate concentrations remain below the Canadian Water Quality Guideline of 3 mg/L.

Table 61: Predicted Nitrate Concentrations at Assessment Points by Season and Discharge Location

A1 – Welland River East at Triangle Island	Winter	Spring	Summer	Fall	Annual
New WWTP Flow (m^3/s)	0.35				
New WWTP Nitrate Limit	20 mg/L				
A1 – Welland River East at Triangle Island					
Existing Concentration (mg/L) – No Discharge	2.29	1.11	0.49	1.05	1.19
Future Concentration (mg/L) – Discharge at L1	2.81	1.63	0.97	1.54	1.69
Future Concentration (mg/L) – Discharge at L2	2.29	1.11	0.49	1.05	1.19
Future Concentration (mg/L) – Discharge at L3	2.29	1.11	0.49	1.05	1.19
Future Concentration (mg/L) – Discharge at L4	2.29	1.11	0.49	1.05	1.19
A2 – HEPC at Montrose Gate					
Existing Concentration (mg/L) – No Discharge	0.37	0.34	0.27	0.21	0.30
Future Concentration (mg/L) – Discharge at L1	0.39	0.36	0.29	0.23	0.32
Future Concentration (mg/L) – Discharge at L2	0.39	0.36	0.29	0.23	0.32
Future Concentration (mg/L) – Discharge at L3	0.39	0.36	0.29	0.23	0.32
Future Concentration (mg/L) – Discharge at L4	0.37	0.34	0.27	0.21	0.30
A3 – Chippewa Creek at Triangle Island					
Existing Concentration (mg/L) – No Discharge	0.31	0.31	0.26	0.18	0.27
Future Concentration (mg/L) – Discharge at L1	0.31	0.31	0.26	0.18	0.27
Future Concentration (mg/L) – Discharge at L2	0.31	0.31	0.26	0.18	0.27
Future Concentration (mg/L) – Discharge at L3	0.33	0.33	0.28	0.20	0.29
Future Concentration (mg/L) – Discharge at L4	0.31	0.31	0.26	0.18	0.27
A4 – Niagara River at Falls (Canadian Shore)					
Existing Concentration (mg/L) – No Discharge	0.31	0.31	0.26	0.18	0.27
Future Concentration (mg/L) – Discharge at L1	0.31	0.31	0.26	0.18	0.27
Future Concentration (mg/L) – Discharge at L2	0.31	0.31	0.26	0.18	0.27
Future Concentration (mg/L) – Discharge at L3	0.31	0.31	0.26	0.18	0.27
Future Concentration (mg/L) – Discharge at L4	0.42	0.41	0.37	0.29	0.37
A5 – Niagara River below Sir Adam Beck GS					
Existing Concentration (mg/L) – No Discharge	0.40	0.36	0.29	0.23	0.32
Future Concentration (mg/L) – Discharge at L1	0.42	0.38	0.31	0.25	0.34
Future Concentration (mg/L) – Discharge at L2	0.42	0.38	0.31	0.25	0.34
Future Concentration (mg/L) – Discharge at L3	0.42	0.38	0.31	0.25	0.34
Future Concentration (mg/L) – Discharge at L4	0.40	0.36	0.29	0.23	0.32
A6 – Niagara River below Sir Adam Beck GS					
Existing Concentration (mg/L) – No Discharge	0.32	0.32	0.26	0.19	0.27
Future Concentration (mg/L) – Discharge at L1	0.32	0.32	0.27	0.19	0.27
Future Concentration (mg/L) – Discharge at L2	0.32	0.32	0.27	0.19	0.27
Future Concentration (mg/L) – Discharge at L3	0.32	0.32	0.27	0.19	0.27
Future Concentration (mg/L) – Discharge at L4	0.32	0.32	0.27	0.19	0.27

5.5 *E. coli*

Table 62 compares the water quality effects of proposed discharge location alternatives at each of six assessment points with the conventional secondary treatment with disinfection effluent limit for *E. coli* (200 cfu/100ml) being applied consistently across seasons and locations.

While the tabulated provide some insight into potential changes in *E. coli* concentrations it should be noted that there are no water quality concerns as the effluent objectives meet provincial guidelines for receiving water quality.

Table 62: Predicted *E. coli* Concentrations at Assessment Points by Season and Discharge Location

A1 – Welland River East at Triangle Island	Winter	Spring	Summer	Fall	Annual
New WWTP Flow (m^3/s)	0.35				
New WWTP *E. coli* Limit	200 cfu/100 mL				
A1 – Welland River East at Triangle Island					
Existing Concentration (cfu/100 mL) – No Discharge	6920.0	308.0	105.0	170.0	1695.1
Future Concentration (cfu/100 mL) – Discharge at L1	6721.2	305.0	107.4	170.8	1654.9
Future Concentration (cfu/100 mL) – Discharge at L2	6920.0	308.0	105.0	170.0	1695.1
Future Concentration (cfu/100 mL) – Discharge at L3	6920.0	308.0	105.0	170.0	1695.1
Future Concentration (cfu/100 mL) – Discharge at L4	6920.0	308.0	105.0	170.0	1695.1
A2 – HEPC at Montrose Gate					
Existing Concentration (cfu/100 mL) – No Discharge	274.2	22.4	11.8	31.4	84.1
Future Concentration (cfu/100 mL) – Discharge at L1	274.2	22.6	12.0	31.6	84.2
Future Concentration (cfu/100 mL) – Discharge at L2	274.2	22.6	12.0	31.6	84.2
Future Concentration (cfu/100 mL) – Discharge at L3	274.2	22.6	12.0	31.6	84.2
Future Concentration (cfu/100 mL) – Discharge at L4	274.2	22.4	11.8	31.4	84.1
A3 – Chippewa Creek at Triangle Island					
Existing Concentration (cfu/100 mL) – No Discharge	50.2	12.1	8.0	26.1	24.0
Future Concentration (cfu/100 mL) – Discharge at L1	50.2	12.1	8.0	26.1	24.0
Future Concentration (cfu/100 mL) – Discharge at L2	50.2	12.1	8.0	26.1	24.0
Future Concentration (cfu/100 mL) – Discharge at L3	50.4	12.3	8.2	26.2	24.2
Future Concentration (cfu/100 mL) – Discharge at L4	50.2	12.1	8.0	26.1	24.0
A4 – Niagara River at Falls (Canadian Shore)					
Existing Concentration (cfu/100 mL) – No Discharge	50.0	12.0	8.0	26.0	23.7
Future Concentration (cfu/100 mL) – Discharge at L1	50.0	12.0	8.0	26.0	23.7
Future Concentration (cfu/100 mL) – Discharge at L2	50.0	12.0	8.0	26.0	23.7
Future Concentration (cfu/100 mL) – Discharge at L3	50.0	12.0	8.0	26.0	23.7
Future Concentration (cfu/100 mL) – Discharge at L4	51.1	13.0	9.1	27.1	24.8
A5 – Niagara River below Sir Adam Beck GS					
Existing Concentration (cfu/100 mL) – No Discharge	274.1	22.8	12.3	31.8	84.3
Future Concentration (cfu/100 mL) – Discharge at L1	274.0	23.0	12.4	31.9	84.5
Future Concentration (cfu/100 mL) – Discharge at L2	274.0	23.0	12.4	31.9	84.5
Future Concentration (cfu/100 mL) – Discharge at L3	274.0	23.0	12.4	31.9	84.5
Future Concentration (cfu/100 mL) – Discharge at L4	274.1	22.8	12.3	31.8	84.3
A6 – Niagara River below Sir Adam Beck GS					
Existing Concentration (cfu/100 mL) – No Discharge	77.8	13.3	8.5	26.7	31.2
Future Concentration (cfu/100 mL) – Discharge at L1	77.8	13.3	8.6	26.7	31.2
Future Concentration (cfu/100 mL) – Discharge at L2	77.8	13.3	8.6	26.7	31.2
Future Concentration (cfu/100 mL) – Discharge at L3	77.8	13.3	8.6	26.7	31.2
Future Concentration (cfu/100 mL) – Discharge at L4	77.8	13.3	8.6	26.7	31.2

5.6 Biological Oxygen Demand ($CBOD_5$)

Table 63 compares the water quality effects of proposed discharge location alternatives at each of six assessment points with the conventional secondary treatment effluent limit for $CBOD_5$ (25 mg/L) being applied consistently across seasons and locations.

While the tabulated provide some insight into potential changes $CBOD_5$ concentrations it should be noted that there are no water quality concerns as the effluent objectives meet provincial guidelines for receiving water quality.

Table 63: Predicted CBOD5 Concentrations at Assessment Points by Season and Discharge Location

A1 – Welland River East at Triangle Island	Winter	Spring	Summer	Fall	Annual
New WWTP Flow (m^3/s)	0.35				
New WWTP $CBOD_5$ Limit	25 mg/L				
A1 – Welland River East at Triangle Island					
Existing Concentration - No Discharge (mg/L)	1.34	1.03	2.00	1.00	1.36
Future Concentration - Discharge at L1 (m/L)	2.04	1.69	2.57	1.63	1.99
Future Concentration - Discharge at L2 (mg/L)	1.34	1.03	2.00	1.00	1.36
Future Concentration - Discharge at L3 (mg/L)	1.34	1.03	2.00	1.00	1.36
Future Concentration - Discharge at L4 (mg/L)	1.34	1.03	2.00	1.00	1.36
A2 - HEPC at Montrose Gate					
Existing Concentration - No Discharge (mg/L)	1.98	1.97	2.00	1.96	1.98
Future Concentration - Discharge at L1 (m/L)	2.00	1.99	2.02	1.99	2.00
Future Concentration - Discharge at L2 (mg/L)	2.00	1.99	2.02	1.99	2.00
Future Concentration - Discharge at L3 (mg/L)	2.00	1.99	2.02	1.99	2.00
Future Concentration - Discharge at L4 (mg/L)	1.98	1.97	2.00	1.96	1.98
A3 - Chippewa Creek at Triangle Island					
Existing Concentration - No Discharge (mg/L)	2.00	2.00	2.00	2.00	2.00
Future Concentration - Discharge at L1 (m/L)	2.00	2.00	2.00	2.00	2.00
Future Concentration - Discharge at L2 (mg/L)	2.00	2.00	2.00	2.00	2.00
Future Concentration - Discharge at L3 (mg/L)	2.02	2.02	2.02	2.02	2.02
Future Concentration - Discharge at L4 (mg/L)	2.00	2.00	2.00	2.00	2.00
A4 - Niagara River at Falls (Canadian Shore)					
Existing Concentration - No Discharge (mg/L)	2.00	2.00	2.00	2.00	2.00
Future Concentration - Discharge at L1 (m/L)	2.00	2.00	2.00	2.00	2.00
Future Concentration - Discharge at L2 (mg/L)	2.00	2.00	2.00	2.00	2.00
Future Concentration - Discharge at L3 (mg/L)	2.00	2.00	2.00	2.00	2.00
Future Concentration - Discharge at L4 (mg/L)	2.14	2.13	2.14	2.14	2.13
A5 - HEPC at Sir Adam Beck					
Existing Concentration - No Discharge (mg/L)	2.03	2.02	2.05	2.01	2.03
Future Concentration - Discharge at L1 (m/L)	2.05	2.04	2.07	2.04	2.05
Future Concentration - Discharge at L2 (mg/L)	2.05	2.04	2.07	2.04	2.05
Future Concentration - Discharge at L3 (mg/L)	2.05	2.04	2.07	2.04	2.05
Future Concentration - Discharge at L4 (mg/L)	2.03	2.02	2.05	2.01	2.03
A6 - Niagara River Below Sir Adam Beck					
Existing Concentration - No Discharge (mg/L)	2.00	2.00	2.01	2.00	2.00
Future Concentration - Discharge at L1 (m/L)	2.01	2.00	2.01	2.00	2.01
Future Concentration - Discharge at L2 (mg/L)	2.01	2.00	2.01	2.00	2.01
Future Concentration - Discharge at L3 (mg/L)	2.01	2.00	2.01	2.00	2.01
Future Concentration - Discharge at L4 (mg/L)	2.01	2.00	2.01	2.00	2.01

6.0 CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Based on the analysis in this report, the following conclusions are provided:

- Elevated total phosphorus concentrations in the Niagara River leads to effluent constraints during the winter for discharges to the HEPC, Chippewa Creek, and the Niagara River.
- Degraded water quality in the Welland River East leads to effluent constraints related to total phosphorus and unionized ammonia for the option to discharge to the Welland River East.
- In most cases, the recommended effluent limits and limits for total and unionized ammonia are defined by the end-of-pipe acute toxicity criteria for unionized ammonia (0.1 mg/L).
- Based on seasonal water temperatures and pH in the receiving water, summer is the most restrictive season for total ammonia. Maximum allowable total ammonia concentrations range from 0.19 mg/L for the Welland River East discharge to 1.0 mg/L for the Chippewa Creek and Niagara River discharges. A value of 0.50 mg/L has been recommended for the Welland River East based on the limits reliably achievable in a nitrifying facility.
- For all other parameters (nitrate, *E. coli*, $CBOD_5$, dissolved oxygen, and TSS) the maximum allowable effluent concentrations at the local and system compliance points are greater than the expected effluent concentrations from a conventional activated sludge treatment plant.
- At most locations and discharge options, the expected water quality concentrations are not expected to be measurably different from the existing conditions. Only the discharge at Location 1 – Welland River East is expected to cause measurable differences in water quality in the immediate area of the discharge.
- Since the modelling presented in this study assumes complete and instant mixing of the effluent after release into the environment, a mixing zone study is required to assess and identify any limitations on assimilative capacity near the outfall.
- Since the information regarding the expected effluent quality from various treatment technologies is not site specific, more detailed assessments should be completed prior to the final selection of the required technology for each discharge location.

Recommendations

Based on the analysis in this report, the recommended effluent objectives and limits for each discharge location are provided in Table 64 through Table 67. Limits and objectives have not been included for nitrate and dissolved oxygen since the effluent quality from any typical plant is expected to be better than the allowable maximum effluent concentrations.

These recommended limits and limits should be re-evaluated upon the completion of a mixing zone study and an assessment of the expected effluent quality from various treatment technologies based in site specific conditions.

Table 64: Proposed Effluent Objectives and Limits for Discharge at Location 1 – Welland River East

Parameter		Proposed Effluent Objectives	Proposed Effluent Limits
Total Phosphorus (mg/L)		0.075	0.100
Total Ammonia (mg/L)	Summer	0.50	0.50
	Winter/Spring/Fall	1.40	1.40
E. coli (cfu/100 mL)		100	200
$CBOD_5$ (mg/L)		15	25
Total Suspended Solids (mg/L)		5	10

Table 65: Proposed Effluent Objectives and Limits for Discharge at Location 2 – Hydro Electric Power Canal

Parameter		Proposed Effluent Objectives	Proposed Effluent Limits
Total Phosphorus (mg/L)		0.5	0.75
Total Ammonia (mg/L)	Summer	1.3	1.3
	Winter/Spring/Fall	2.0	2.0
E. coli (cfu/100 mL)		100	200
$CBOD_5$ (mg/L)		15	25
Total Suspended Solids (mg/L)		15	25

Table 66: Proposed Effluent Objectives and Limits for Discharge at Location 3 – Chippewa Creek

Parameter		Proposed Effluent Objectives	Proposed Effluent Limits
Total Phosphorus (mg/L)		0.5	0.75
Total Ammonia (mg/L)	Summer	1.0	1.0
	Winter/Spring/Fall	1.7	1.7
E. coli (cfu/100 mL)		100	200
$CBOD_5$ (mg/L)		15	25
Total Suspended Solids (mg/L)		15	25

Table 67: Proposed Effluent Objectives and Limits for Discharge at Location 4 – Niagara River

Parameter		Proposed Effluent Objectives	Proposed Effluent Limits
Total Phosphorus (mg/L)		0.5	0.75
Total Ammonia (mg/L)	Summer	1.0	1.0
	Winter/Spring/Fall	1.7	1.7
E. coli (cfu/100 mL)		100	200
$CBOD_5$ (mg/L)		15	25
Total Suspended Solids (mg/L)		15	25

7.0 LIMITATIONS

Golder has prepared this report for the exclusive use by the Niagara Region and other members of the project team for the South Niagara Falls Wastewater Solutions Schedule C Class EA Project. The results presented in this report are for a proposed wastewater treatment plant with a specific design capacity of 30 MLD discharging to four potential locations in the study area. The results presented in this report should not be used to assess other design capacities or discharge locations in any way.

Information, analysis, and commentary presented in this report regarding wastewater treatment technologies and the associated typical effluent quality have been provided by CIMA+.

The assessment has been completed using data and information collected and provided by others. Golder does not assume any responsibility related to the accuracy or reliability of the data or information.

Water quality modelling requires the use of many assumptions due to the uncertainty related to determining the physical and chemical characteristics of a complex system. The prediction of water quality is based on several inputs (flows and chemistry), all of which have inherent variability and uncertainty.

GoldSim derives a maximum allowable concentration distribution for each parameter and location by combining randomly sampled flows over numerous (1,000s) of cycles using a Monte Carlo approach. While this approach is valuable because it considers numerous combinations, it may be inaccurate if certain environmental conditions are less represented in historic data than others.

The conventional mass balance ACS approach calculates the maximum allowable effluent concentration for a specific case where the low-flow condition (e.g., 7Q20) occurs for all the inflows at the same time. This is the approach that is typically requested by the MECP and is assumed to represent a worst-case scenario. However, because of the range of the inflow watershed sizes (e.g., Niagara River compared to Lyons Creek), it is highly unlikely that low-flow conditions will occur in all the inflows at the same time.

In natural systems and complex man-made systems, observed conditions will almost certainly vary with respect to estimated conditions. Water quality and flow data has shown a vast range of variability across seasons and locations. This variability may not be captured by the flow and water quality statistics (e.g., 75th percentile concentrations) used as inputs to the models. This is especially true for data sets with small sample sizes.

The modelling presented in this study assumes complete and instant mixing of the effluent after release into the environment. As such, this assessment does not consider any potential water quality effects in the immediate area of the outfall. A mixing zone study is required to assess these issues and identify any related limitations on assimilative capacity near the outfall.

Since the information regarding the expected effluent quality from various treatment technologies is not site specific, more detailed assessments should be completed prior to the final selection of the required technology for each discharge location.

This assessment is one part of a larger project to select the location and effluent criteria for the proposed wastewater treatment plant. The results of this assessment should be used in conjunction with the other components of the Project to support any decisions. Given all the inherent uncertainties provided, the results should be used as a tool to aid in the design and planning of the proposed wastewater treatment plant rather than to provide absolute water quality predictions.

Signature Page

We trust that this report meets your needs at this time. If you have any questions, please do not hesitate to contact the undersigned.

Yours truly,

CIMA+

Troy Briggs, MEng, PEng
Manager, Wastewater

Golder Associates Ltd.

Marta Lopez-Egea, MASc
Water Resources Specialist

Gerard van Arkel, MEng, PEng
Associate, Senior Water Resources Engineer

TB/MLE/GR/GVA/hp/mp

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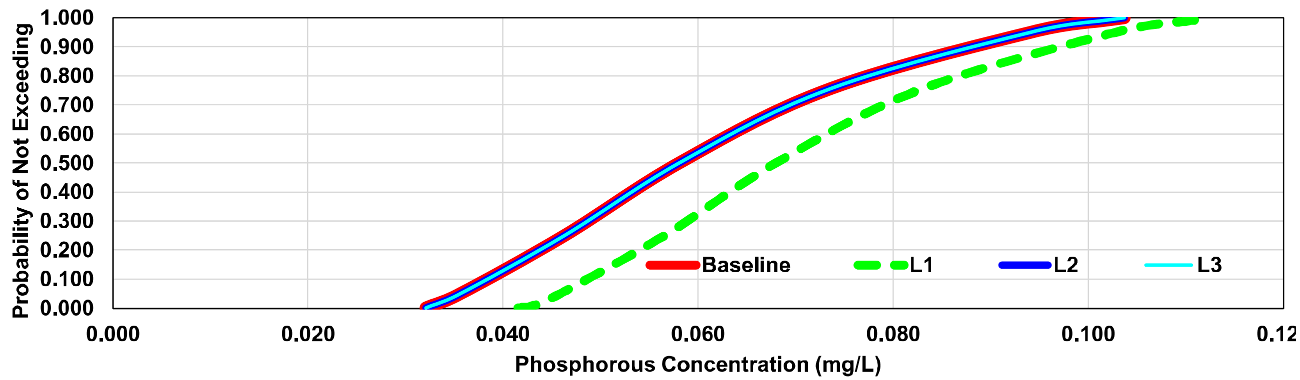
APPENDIX A

Predicted Phosphorus Concentration Distributions in Welland River East, Chippewa Creek, and HEPC

Prediction of Phosphorous for Summer

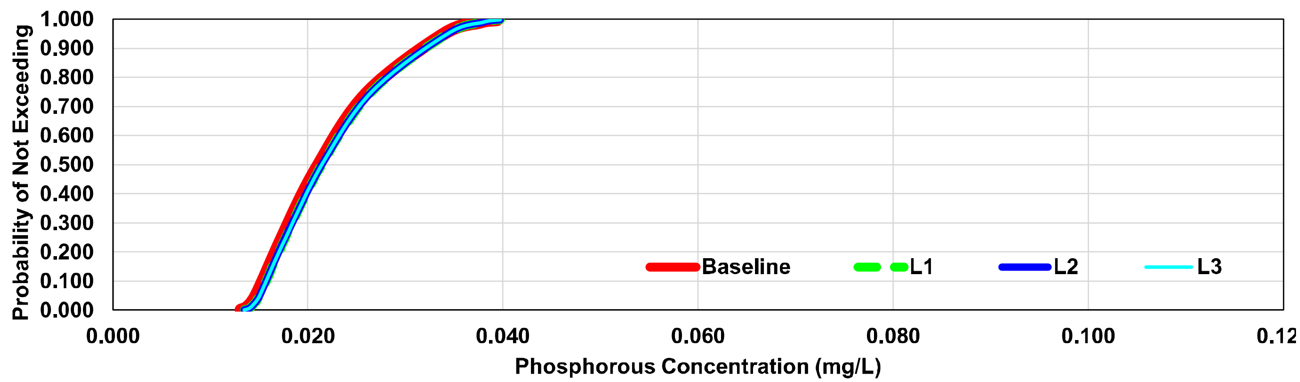
Predictions at L1

Cumulative Probability	Baseline	L1	L2	L3
0.001	0.032	0.042	0.032	0.032
0.010	0.033	0.043	0.033	0.033
0.050	0.036	0.046	0.036	0.036
0.250	0.046	0.057	0.046	0.046
0.500	0.058	0.068	0.058	0.058
0.750	0.073	0.083	0.073	0.073
0.950	0.094	0.103	0.094	0.094
0.990	0.102	0.111	0.102	0.102
0.999	0.104	0.113	0.104	0.104



Predictions at L2

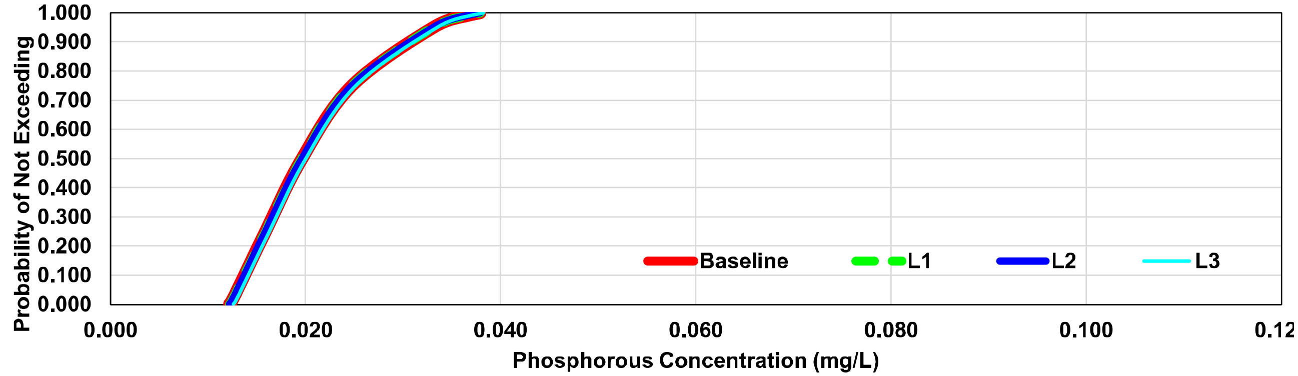
Cumulative Probability	Baseline	L1	L2	L3
0.001	0.013	0.014	0.014	0.014
0.010	0.014	0.014	0.014	0.014
0.050	0.015	0.015	0.015	0.015
0.250	0.017	0.018	0.018	0.018
0.500	0.021	0.021	0.021	0.021
0.750	0.026	0.027	0.027	0.027
0.950	0.034	0.034	0.034	0.034
0.990	0.038	0.038	0.038	0.038
0.999	0.039	0.040	0.040	0.040



Prediction of Phosphorous for Summer

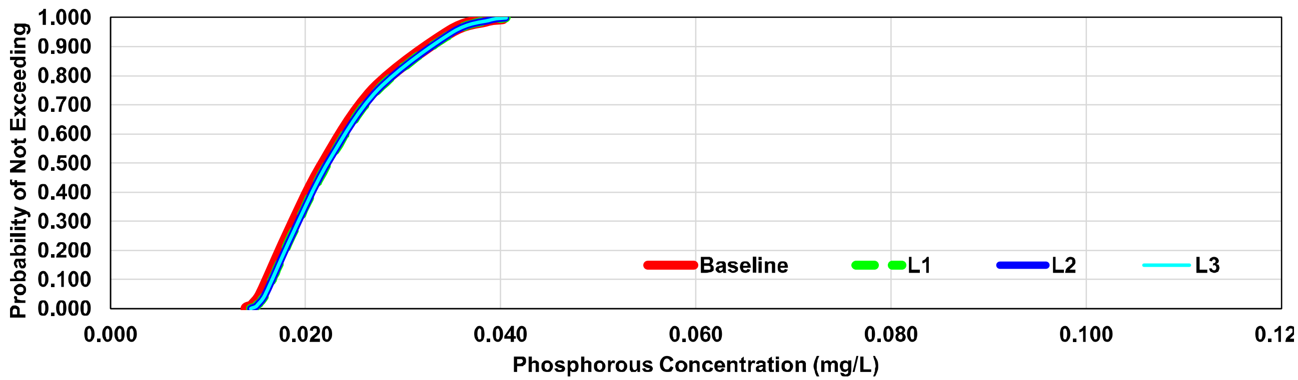
Predictions at L3

Cumulative Probability	Baseline	L1	L2	L3
0.001	0.012	0.012	0.012	0.013
0.010	0.012	0.012	0.012	0.013
0.050	0.013	0.013	0.013	0.013
0.250	0.016	0.016	0.016	0.016
0.500	0.020	0.020	0.020	0.020
0.750	0.025	0.025	0.025	0.025
0.950	0.033	0.033	0.033	0.033
0.990	0.037	0.037	0.037	0.037
0.999	0.038	0.038	0.038	0.038



Predictions at System Compliance Point

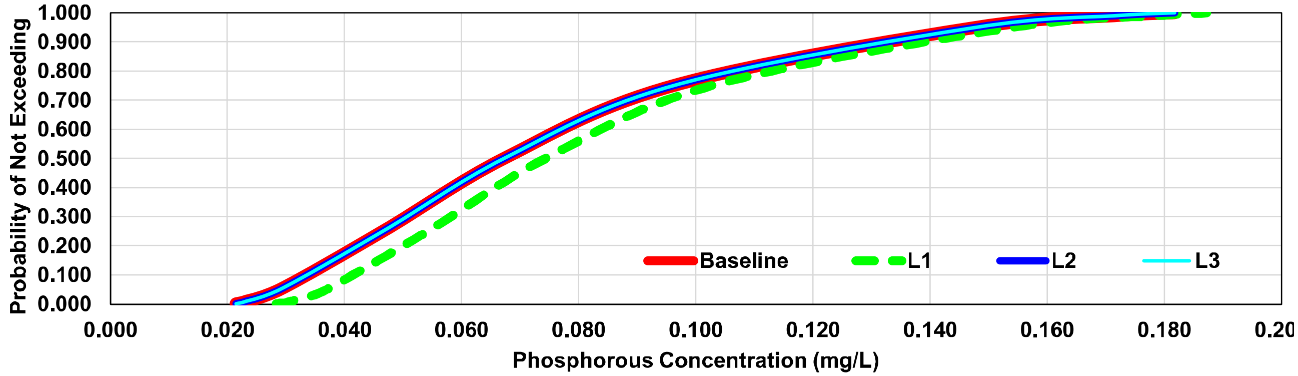
Cumulative Probability	Baseline	L1	L2	L3
0.001	0.014	0.014	0.014	0.014
0.010	0.015	0.015	0.015	0.015
0.050	0.016	0.016	0.016	0.016
0.250	0.018	0.019	0.019	0.019
0.500	0.022	0.022	0.022	0.022
0.750	0.027	0.027	0.027	0.027
0.950	0.035	0.035	0.035	0.035
0.990	0.038	0.039	0.039	0.039
0.999	0.040	0.041	0.041	0.041



Prediction of Phosphorous for Fall

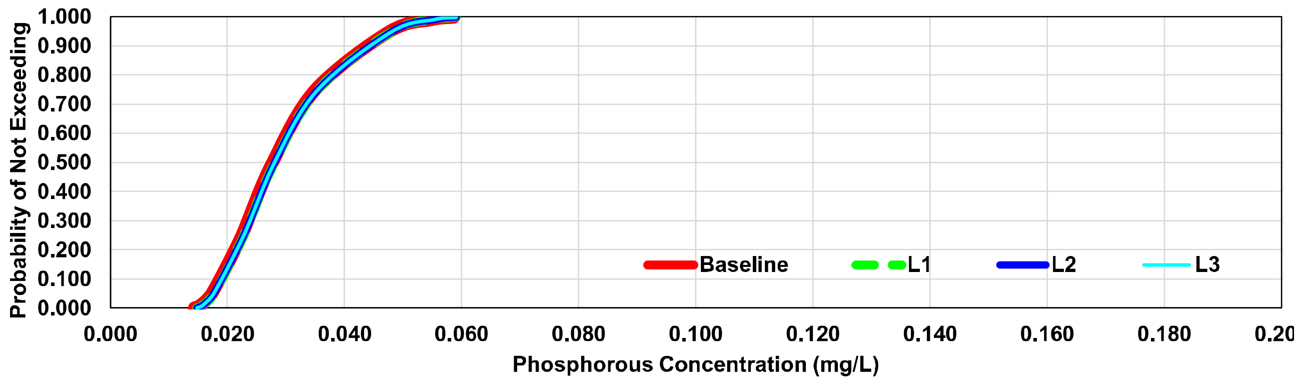
Predictions at L1

Cumulative Probability	Baseline	L1	L2	L3
0.001	0.021	0.028	0.021	0.021
0.010	0.023	0.031	0.023	0.023
0.050	0.029	0.037	0.029	0.029
0.250	0.047	0.054	0.047	0.047
0.500	0.067	0.074	0.067	0.067
0.750	0.096	0.103	0.096	0.096
0.950	0.148	0.154	0.148	0.148
0.990	0.173	0.179	0.173	0.173
0.999	0.182	0.187	0.182	0.182



Predictions at L2

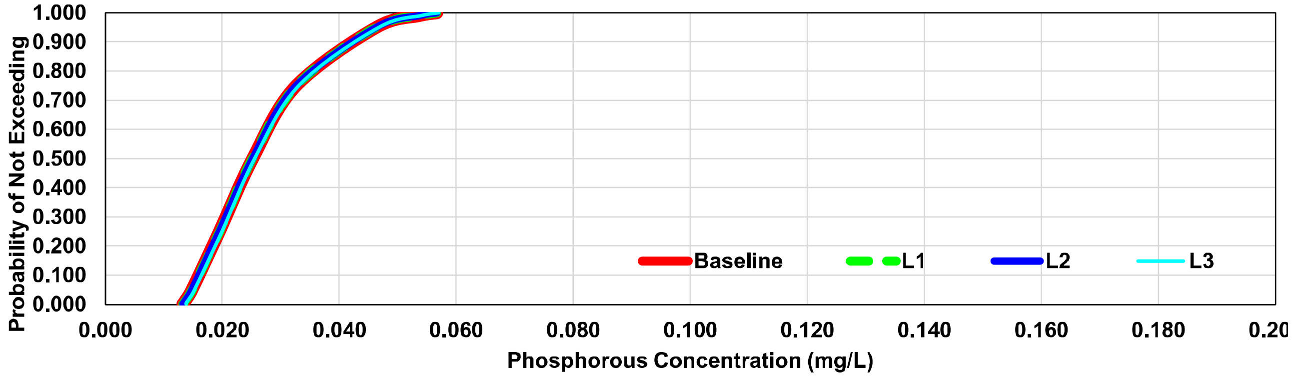
Cumulative Probability	Baseline	L1	L2	L3
0.001	0.014	0.015	0.015	0.015
0.010	0.015	0.016	0.016	0.016
0.050	0.017	0.018	0.018	0.018
0.250	0.022	0.023	0.023	0.023
0.500	0.028	0.028	0.028	0.028
0.750	0.035	0.035	0.035	0.035
0.950	0.048	0.048	0.048	0.048
0.990	0.055	0.055	0.055	0.055
0.999	0.059	0.059	0.059	0.059



Prediction of Phosphorous for Fall

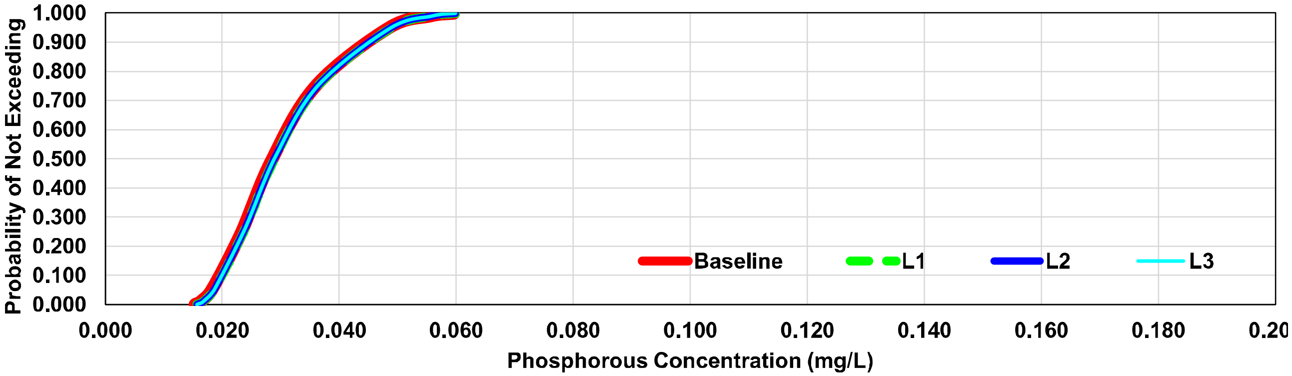
Predictions at L3

Cumulative Probability	Baseline	L1	L2	L3
0.001	0.013	0.013	0.013	0.014
0.010	0.014	0.014	0.014	0.014
0.050	0.015	0.015	0.015	0.015
0.250	0.020	0.020	0.020	0.020
0.500	0.025	0.025	0.025	0.026
0.750	0.033	0.033	0.033	0.033
0.950	0.046	0.046	0.046	0.047
0.990	0.054	0.054	0.054	0.054
0.999	0.057	0.057	0.057	0.057



Predictions at System Compliance Point

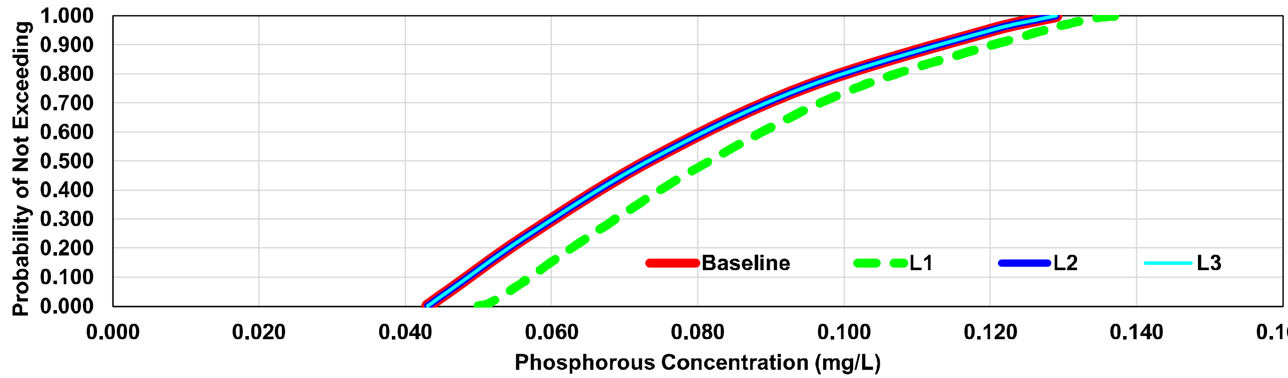
Cumulative Probability	Baseline	L1	L2	L3
0.001	0.015	0.016	0.016	0.016
0.010	0.016	0.017	0.017	0.017
0.050	0.018	0.019	0.019	0.019
0.250	0.023	0.024	0.024	0.024
0.500	0.029	0.029	0.029	0.029
0.750	0.036	0.036	0.036	0.036
0.950	0.049	0.049	0.049	0.049
0.990	0.056	0.056	0.056	0.056
0.999	0.060	0.060	0.060	0.060



Prediction of Phosphorous for Winter

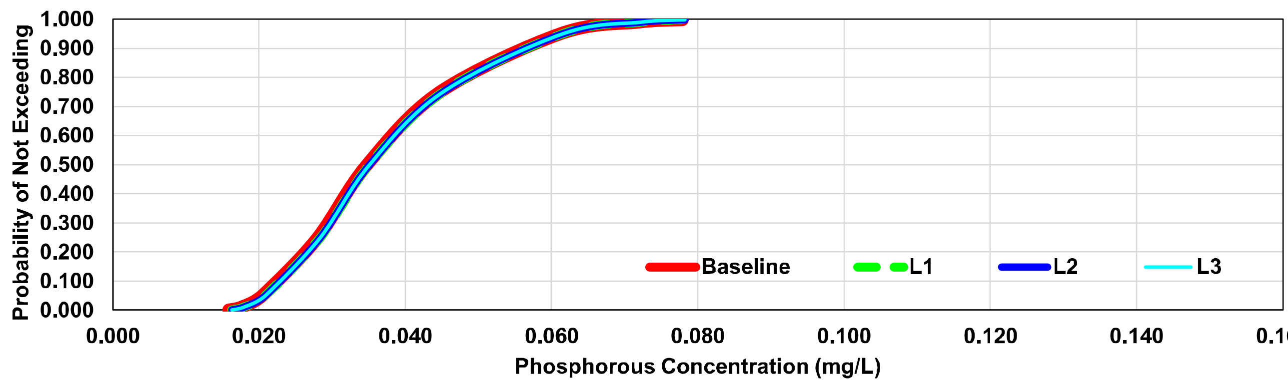
Predictions at L1

Cumulative Probability	Baseline	L1	L2	L3
0.001	0.043	0.050	0.043	0.043
0.010	0.044	0.051	0.044	0.044
0.050	0.046	0.054	0.046	0.046
0.250	0.057	0.066	0.057	0.057
0.500	0.073	0.082	0.073	0.073
0.750	0.094	0.102	0.094	0.094
0.950	0.120	0.127	0.120	0.120
0.990	0.127	0.134	0.127	0.127
0.999	0.129	0.137	0.129	0.129



Predictions at L2

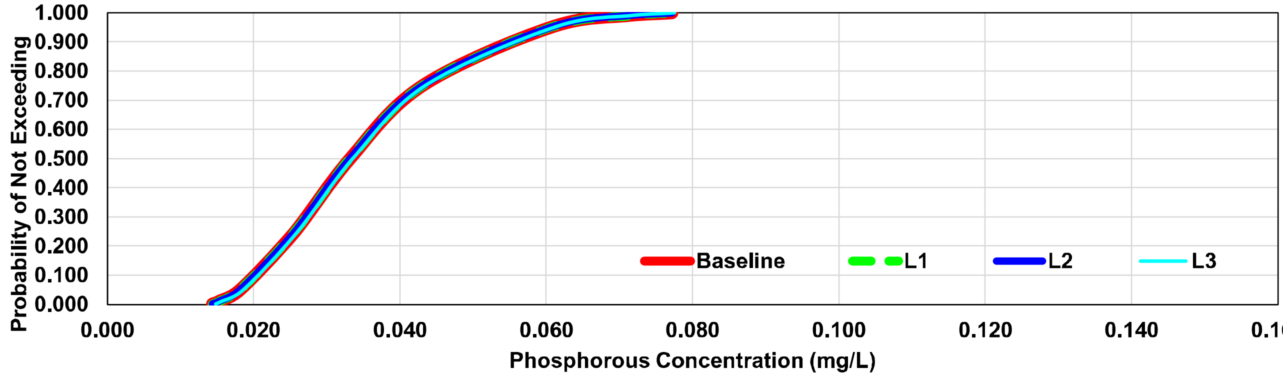
Cumulative Probability	Baseline	L1	L2	L3
0.001	0.016	0.016	0.016	0.016
0.010	0.018	0.018	0.018	0.018
0.050	0.020	0.021	0.021	0.021
0.250	0.028	0.028	0.028	0.028
0.500	0.035	0.035	0.035	0.035
0.750	0.045	0.045	0.045	0.045
0.950	0.061	0.062	0.062	0.062
0.990	0.072	0.072	0.072	0.072
0.999	0.078	0.078	0.078	0.078



Prediction of Phosphorous for Winter

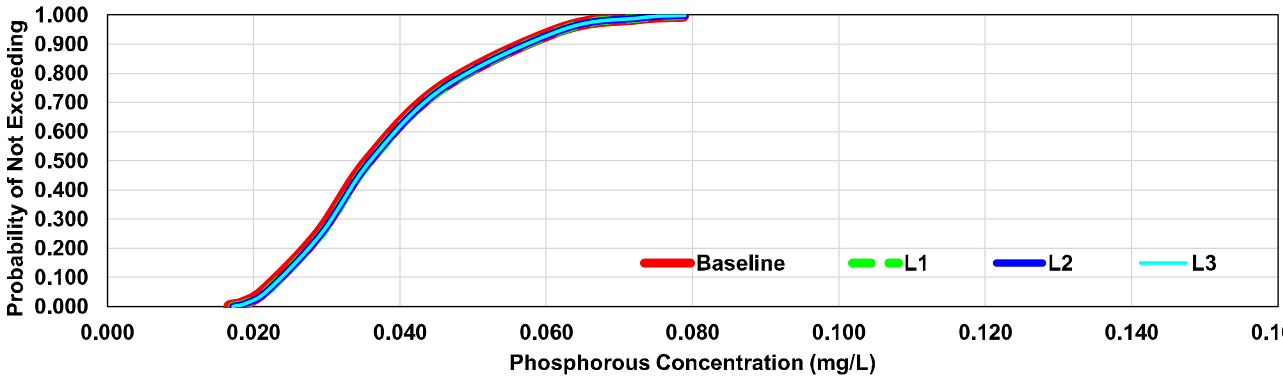
Predictions at L3

Cumulative Probability	Baseline	L1	L2	L3
0.001	0.014	0.014	0.014	0.015
0.010	0.015	0.015	0.015	0.016
0.050	0.018	0.018	0.018	0.019
0.250	0.026	0.026	0.026	0.026
0.500	0.033	0.033	0.033	0.034
0.750	0.043	0.043	0.043	0.043
0.950	0.061	0.061	0.061	0.061
0.990	0.071	0.071	0.071	0.072
0.999	0.077	0.077	0.077	0.078



Predictions at System Compliance Point

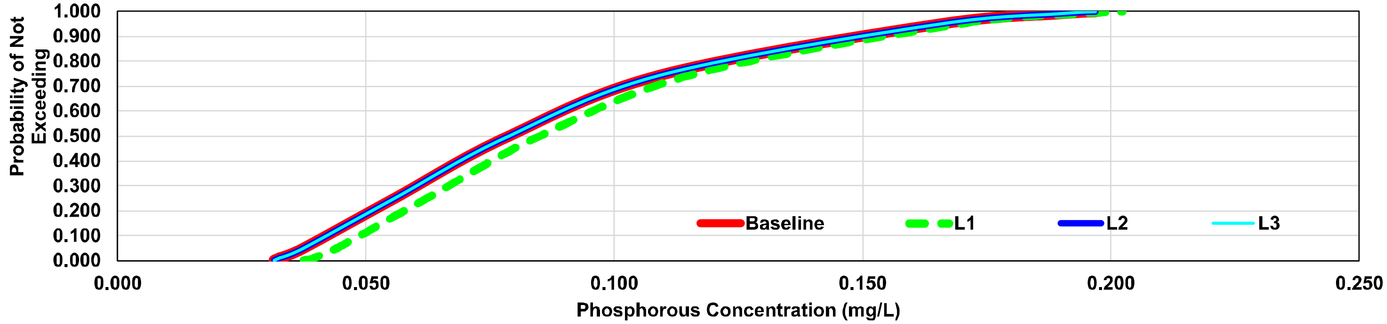
Cumulative Probability	Baseline	L1	L2	L3
0.001	0.017	0.017	0.017	0.017
0.010	0.018	0.019	0.019	0.019
0.050	0.021	0.022	0.022	0.022
0.250	0.029	0.029	0.029	0.029
0.500	0.036	0.036	0.036	0.036
0.750	0.046	0.046	0.046	0.046
0.950	0.062	0.062	0.062	0.062
0.990	0.073	0.073	0.073	0.073
0.999	0.079	0.079	0.079	0.079



Prediction of Phosphorous for Spring

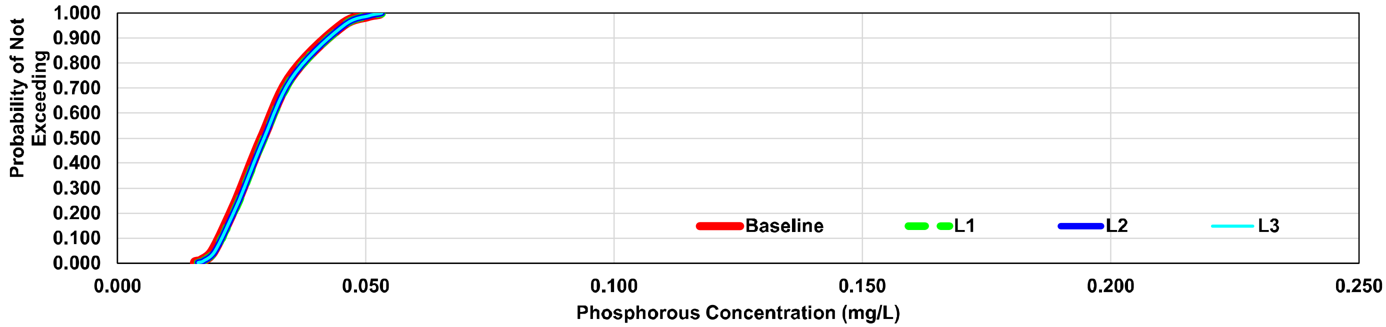
Predictions at L1

Cumulative Probability	Baseline	L1	L2	L3
0.001	0.032	0.037	0.032	0.032
0.010	0.033	0.039	0.033	0.033
0.050	0.038	0.044	0.038	0.038
0.250	0.056	0.062	0.056	0.056
0.500	0.079	0.085	0.079	0.079
0.750	0.110	0.116	0.110	0.110
0.950	0.166	0.170	0.166	0.166
0.990	0.190	0.194	0.190	0.190
0.999	0.197	0.202	0.197	0.197



Predictions at L2

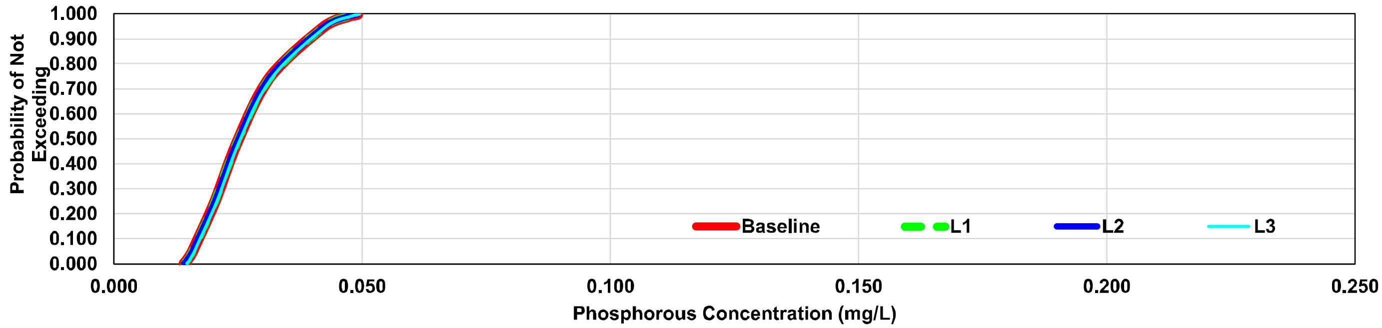
Cumulative Probability	Baseline	L1	L2	L3
0.001	0.016	0.016	0.016	0.016
0.010	0.017	0.017	0.017	0.017
0.050	0.019	0.020	0.020	0.020
0.250	0.024	0.024	0.024	0.024
0.500	0.029	0.030	0.030	0.030
0.750	0.035	0.035	0.035	0.035
0.950	0.045	0.045	0.045	0.045
0.990	0.050	0.051	0.051	0.051
0.999	0.053	0.053	0.053	0.053



Prediction of Phosphorous for Spring

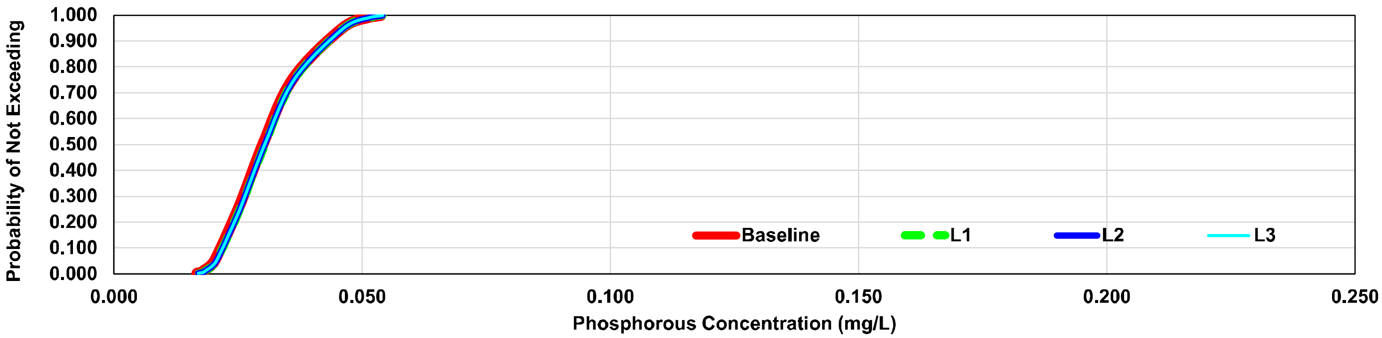
Predictions at L3

Cumulative Probability	Baseline	L1	L2	L3
0.001	0.014	0.014	0.014	0.015
0.010	0.015	0.015	0.015	0.015
0.050	0.016	0.016	0.016	0.016
0.250	0.020	0.020	0.020	0.021
0.500	0.025	0.025	0.025	0.025
0.750	0.032	0.032	0.032	0.032
0.950	0.042	0.042	0.042	0.043
0.990	0.047	0.047	0.047	0.048
0.999	0.049	0.049	0.049	0.049



Predictions at System Compliance Point

Cumulative Probability	Baseline	L1	L2	L3
0.001	0.017	0.017	0.017	0.017
0.010	0.018	0.018	0.018	0.018
0.050	0.020	0.021	0.021	0.021
0.250	0.025	0.025	0.025	0.025
0.500	0.030	0.030	0.030	0.030
0.750	0.036	0.036	0.036	0.036
0.950	0.046	0.046	0.046	0.046
0.990	0.051	0.052	0.052	0.052
0.999	0.054	0.054	0.054	0.054



V3.11.2

REGIONAL MUNICIPALITY OF NIAGARA
SOUTH NIAGARA FALLS WASTEWATER SOLUTIONS

WWTP Design Basis

Technology Review – New WWTP

Regional Municipality of Niagara

South Niagara Falls WWTP Class Environment Assessment and Conceptual Design

TM No. 2 Technology Review

March 2022

SUBMITTED BY CIMA CANADA INC.
5935 Airport Road
Mississauga, ON L4V 1W5
T 905 695-1005
cima.ca

Niagara Region

South Niagara Falls WWTP Class Environment Assessment and Conceptual Design
TM No. 2 Technology Review

Project T001140A

PREPARED BY:

Mina Yousif, M.Eng., P.Eng., PMP

VERIFIED BY:

Troy Briggs, M.Eng., P.Eng.

CIMA+
5935 Airport Road, Unit 500
Mississauga, Ontario L4V 1W5

March 2022

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1 Introduction

1.1 Background

The Regional Municipality of Niagara (Niagara Region) completed a Water and Wastewater Master Servicing Plan (Master Plan) in 2017 that provided a long-term planning strategy to address the water and wastewater system needs to the year 2041 (GM BluePlan, 2017). The Master Plan recommended a combination of solutions for meeting future needs, including improving the existing sewage collection systems, and construction of a new wastewater treatment plant (named South Niagara Falls WWTP) to service growth in south Niagara Falls in two stages:

- Stage 1: Provide a capacity of 30 megaliters per day (MLD), including approximately 15 MLD from the existing Niagara Falls WWTP, which currently services the existing developed South Niagara Falls area, and approximately 15 MLD from new growth in that area;
- Stage 2: Provide a capacity increase to 60 MLD to accommodate future servicing to full build-out capacity.

The 2017 Master Plan was completed under the Environmental Assessment Act in accordance with Phases 1 and 2 of Municipal Class Environmental Assessment (EA) requirements (2000, as amended in 2007, 2011 and 2015). The Master Plan concluded that a Schedule “C” Class EA study is required to address Phases 3 and 4 requirements of the Municipal Class EA planning process.

GM BluePlan, in association with CIMA+, has been retained by the Region to complete the Schedule “C” Class EA study and Conceptual Design for the proposed South Niagara Falls WWTP (SNF WWTP). The Class EA study will present development and evaluation of alternative design concepts for the preferred solution including their associated environmental impacts and proposed mitigation measures.

This technical memorandum (TM No. 2) has been prepared to develop and evaluate alternative design concepts for the proposed SNF WWTP. The purpose is to provide the treatment process unit selection of the facility.

1.2 Purpose of TM No. 2

The purpose of TM No. 2 is to:

- Identify and develop a long list of treatment technology alternatives for each unit process of both liquid and solids trains
- Provide screening of long-list alternatives to produce a short list of technology alternatives for further evaluation
- Evaluate the short-listed alternatives based on a set of evaluation criteria
- Recommend preferred treatment technology alternatives for each unit process

2 Design Basis

This section provides a summary of design basis for the evaluation of all the treatment technology alternatives for the proposed SNF WWTP, as developed in TM No. 1 – Design Basis.

2.1 Design Flows

Table 1 summarizes the proposed design average and peak flows for the proposed SNF WWTP.

Table 1 Proposed Design Flows for the SNF WWTP

Design Parameter	New Plant	Peaking Factor
Average Day Flow (ADF)	30 MLD	-
Maximum Month Flow (MMF)	40 MLD	1.3
Maximum Day Flow (MDF)	76 MLD	2.5
Peak Hourly Flow (PHF)	106 MLD	3.5
Peak Instantaneous Flow (PIF)	120 MLD	4.0

2.2 Design Loadings

The design of the SNF WWTP will be based on the combined raw sewage, centrate flows from the Garner Road Biosolids Facility, and hauled waste. The plant will accommodate the incremental hydraulic, solids, and organic and nutrient loads imposed from the external recycle stream on the plant. A plant-wide specific mass balance will be developed for the preferred treatment train as part of the Conceptual Design.

The existing Niagara Falls WWTP historical flow and loading data from 2017 to 2020, along with the Garner Road Biosolids Facility historical centrate data and typical hauled waste concentrations, were reviewed and statistically analyzed in TM No. 1 to develop the design basis for the proposed SNF WWTP. The recommended design influent loadings for biochemical oxygen demand (BOD_5), total suspended solids (TSS), total phosphorus (TP) and total Kjeldahl nitrogen (TKN) are summarized in Table 2.

Table 2 Recommended Design Loadings for the SNF WWTP

Parameter	Value	Basis
Influent Average Concentration (mg/L)		
BOD_5	330	Average load divided by the average flow
TSS	460	
TP	11	
TKN	90	

Parameter	Value	Basis
Influent Average Loading (kg/d) BOD_5 TSS TP TKN	 10,450 14,570 350 2,885	Typical per capital load (Metcalf & Eddy, 2003), plus centrate loadings from Garner Road Biosolids Facility, and hauled waste
Influent Peak Month Loading (kg/d) BOD_5 TSS TP TKN	 13,600 19,000 500 3,800	Typical peak month loading factor of 1.3

2.3 Effluent Criteria

As part of the Class EA process, an Assimilative Capacity Study (ACS) has recently completed to develop effluent criteria of the proposed South Niagara Falls WWTP which will discharge to Chippewa Creek. The recommended effluent criteria are presented in Table 3. The new plant will have the following requirements:

- Year-round nitrification, to achieve non-toxicity effluent with respect to ammonia, with the current objective as 0.1 mg/L of un-ionized ammonia;
- Non-toxicity effluent with respect to chlorine residual.

Table 3 Design Effluent Objectives and Limits for the SNF WWTP

Parameters	Effluent Objectives (mg/L) (1)	Effluent Limits (mg/L) (1)
Carbonaceous Biochemical Oxygen Demand ($CBOD_5$)	15	25
TSS	15	25
TP	0.5	0.75
Total Ammonia Nitrogen (TAN) May to October November to April	 6.5 12.0	 8.8 15.0
E. Coli (CFU/ 100 mL) (2)	200	200

Notes:
(1) Based on monthly average concentrations.
(2) Based on monthly geometric mean.

It is important to note that based on the results of the ACS, tertiary treatment will not be required. The effluent phosphorous concentrations produced by the secondary treatment alternatives will satisfy the Provincial Water Quality Objectives (PWQO's) and the Ministry of the Environment and Conservation Parks (MECP) requirements for discharge to the receiving water (i.e. Chippewa Creek).

3 Wastewater Treatment Alternatives

3.1 Overview

This section presents the following:

- Identification and development of a number of potentially beneficial long-list technology alternatives for both liquid and solids trains that could be implemented at the proposed SNF WWTP.
- Development of a short-list of the most attractive technology alternatives for the SNF WWTP for further evaluation.

3.2 Long List of Treatment Technologies

The following factors were considered for the identification of the long-list alternatives:

- Flexible and adaptable to changing regulations
- Reliable and proven over the full range of flow and loading conditions
- Simplify long term O&M
- Minimize energy
- Minimize odours

Table 4 provides a long list of wastewater treatment technologies identified for both the liquid and solids trains for the SNF WWTP.

Table 4 Summary of Long List Treatment Technologies

Unit Process	Long List Technologies	Function
Screening	• Mechanically Cleaned Screens (6 mm)	Protects the downstream equipment by removing large debris, assists in maximizing the associated treatment efficiency, and minimizes downstream operational and maintenance issues.
Grit Removal	• Vortex Grit Removal • Aerated Grit Removal	Physically removes heavy, abrasive, inorganic solids from screened wastewater, to protect the downstream equipment from excessive wear, reduce deposit formation in pipes and basins, and reduce solids handling.
Primary Treatment	• Conventional Primary Clarifiers with Separate WAS Thickening • Conventional Primary Clarifiers with Co-thickening	Primary treatment reduces the load on the downstream biological treatment system by removing TSS and BOD_5 and reduce energy consumption.

Secondary Treatment	• Conventional Activated Sludge (CAS) • Moving Bed Biofilm Reactor (MBBR) • Biological Aerated Filter (BAF) • Biological Nutrient Removal (BNR) • Aerobic Granular Sludge (AGS) • Membrane Aerated Biofilm Reactor (MABR)	Removes BOD_5, TSS, suspended and non-settleable colloidal solids, nitrogen, and phosphorous from the wastewater to below acceptable effluent limits.
Disinfection	• Chlorination/Dechlorination • UV Disinfection • Peracetic acid (PAA)	Protects public safety by killing and inactivating pathogens in treated water. Selection of disinfection technologies must also consider impacts on disinfection by-products (DBPs) formation.
WAS Thickening	• Separate WAS thickening • WAS Co-thickening	Reduce sludge volume prior to stabilization and/or dewatering, and final disposal.
Digestion	• Anaerobic Digestion	Provides pathogen reduction, vector attraction reduction, and solids reduction of biosolids prior to final disposal.

The following sections provide a general description of each technology, why it is applicable for the Region's SNF WWTP, how it works, and critical implementation considerations that may impact its applicability.

3.2.1 Screening

A robust and reliable headworks facility is one of the most important unit processes from a hydraulic perspective (flooding risk) and long-term operation and maintenance (O&M) costs. There are several common types of mechanically cleaned screens including:

- Spiral Perforated Screen
- Multi-Rake Bar Screen
- Step Screen
- Travelling Perforated Plate Screen

Among these screens, multi-rake bar screen and step screen are on the Region's Approved Product and Equipment List. As such, only these listed types of screens are described below.

3.2.1.1 Multi-rake Bar Screen

Multi-rake bar screen is a mechanically cleaned bar screen. The screen consists of a stationary bar rack and multiple rakes mounted on the chain in front of the screen, as shown in Figure 1. The installation angle of the multi-rake bar screen can be vertical or at an inclination angle down to 70 degrees. Installing screens at a slight angle provides benefits as follows and is recommended for any new installation:

- Increased screen surface area and hydraulic capacity
- The incline helps rakes carry greater amounts of debris to the surface without the debris falling off the front face of the rake with a vertical (90 degree) installation. This is especially important for screens installed in deep channels and combined sewers which can see significant variations in the screenings and grit volume.

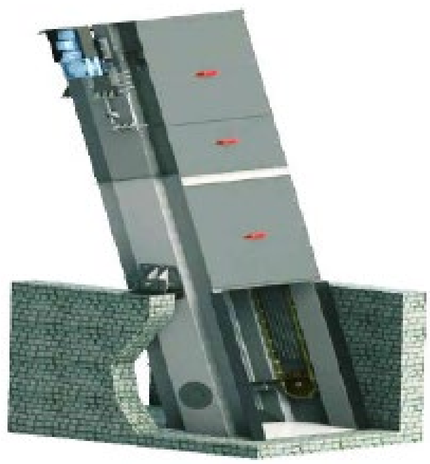

Figure 1 Multi-Rake Bar Screen (Courtesy of Veolia)

The multi-rake bar screen would normally discharge to a separate washer-compactor conveyor leading to the disposal bin. A key advantage of this configuration is that the screen can function even if the washer-compactor is out-of-service by removing the washer-compactor and dropping screenings directly into a bin.

A summary of the advantages and disadvantages of multi-rake bar screens are summarized in Table 5.

Table 5 Summary of Advantages and Disadvantages of Multi-Rake Bar Screen

Advantages	Disadvantages
• Proven technology • Rapid cleaning in event of storm • Self-cleaning, no wash water or brush required • Low maintenance, no lubrication necessary • Low profile above channel floor • High screenings loads • Can operate independent of washer/compactor	• Slightly lower screenings capture rate (single dimensional) compared to perforated plate • High settling risk upstream depending on channel velocities. May require intermittent channel aeration. • Bearing below water level (reduced risk with recent technology advancements)

3.2.1.2 Step Screen

The step screen mechanism consists of a rake assembly that penetrates the screen from behind, and gradually elevates debris that is accumulated on the screen upwards toward the discharge.

This fine screen is comprised of bar spacing to provide the screening. For the step screen, the debris settles on the steps of the screen. The lifting lamella bars penetrate the screen from behind and lift the accumulated debris up towards the next step. This procedure continues until the debris is discarded over the top step. The lifting bars run in a circular motion in order to perform the lift, and beyond the bar assembly, does not have submerged moving parts.

A diagram of the lifting mechanism is shown in Figure 2 below.

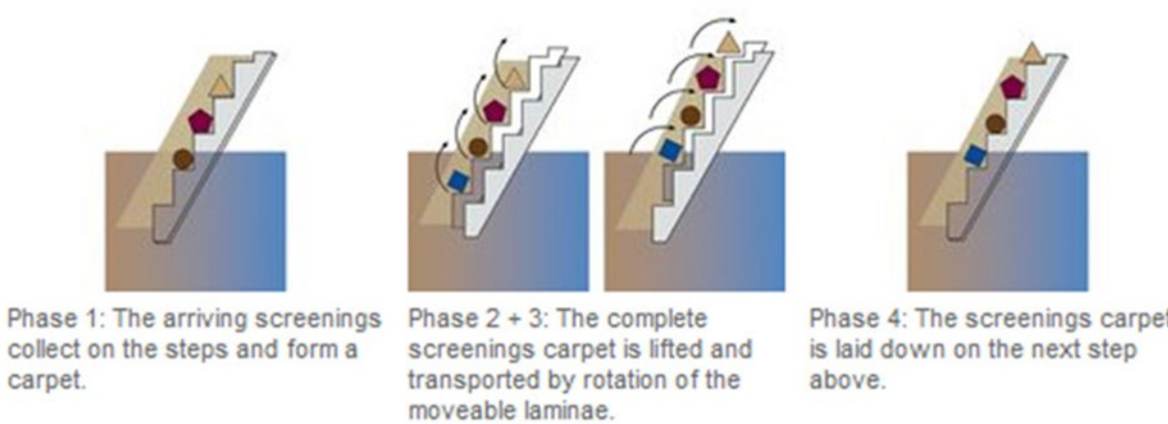

Figure 2 Step Screen Lifting Motion (Courtesy of Huber)

Some of the key considerations for a fine step screen include:

- Guides below the water level can increase maintenance due to grit abrasion/wear;
- Cleaning cycles are rapid as each movement cleans the entire screen face
- Similar screenings removal efficiency as bar screens.
- Cleaning of the entire screen face results in uniformly low channel velocity allowing for grit settlement upstream of the screen. In some installations, this results in increased wear and maintenance.
- Does not require any brushes or fluidizing water for cleaning.

A summary of the advantages and disadvantages of step screens are summarized in Table 6.

Table 6 Summary of Advantages and Disadvantages of Step Screen

Advantages	Disadvantages
• Proven technology • Easy tip out • Self-cleaning, no wash water or brush required • Low profile above channel floor • High screenings loads • Can operate independent of washer/compactor	• Screen inclination of 50% requires the greatest dedicated area for the screens. • Increased maintenance/wear risk with grit settlement in front of screen with wide range of plant flows

To provide the greatest flexibility to the Region, the conceptual design will be based on step screen technology since step screens require the greatest channel area. The preferred screening technology can be confirmed during detailed design.

3.2.2 Grit Removal

Two alternative treatment technologies were considered for grit removal:

- Aerated grit removal
- Vortex grit removal

3.2.2.1 Aerated Grit Removal

An aerated grit removal system consists of aerated grit chambers. In an aerated grit chamber, air is introduced along one side of a rectangular tank to create a spiral flow pattern perpendicular to the flow through the tank. The heavier grit particles that have higher settling velocities settle to the bottom of the tank. Lighter, principally organic, particles remain in suspension and pass through the tank. The velocity of roll or agitation governs the size of particles of a given specific gravity that will be removed.

Grit is removed using a conveyor in the bottom of the tank to feed an external grit slurry pump. The grit slurry is pumped to a grit classifier to separate the grit / water slurry.

A summary of advantages and disadvantages of aerated grit removal is presented in Table 7.

Table 7 Summary of Advantages and Disadvantages of Aerated Grit Removal

Advantages	Disadvantages
• Proven technology • Exhibits consistent removal efficiency over a wide flow range. • Better removal efficiency over vortex grit for systems with heavy grit load	• Moderate energy requirements (due to aeration) • Higher maintenance for grit conveyance equipment • Higher odour release potential with an agitated surface • Larger footprint

3.2.2.2 Vortex Grit Removal

A vortex type grit chamber operates on a similar principal as an aerated grit chamber utilizing a cylindrical tank, which is designed to create a vortex flow pattern. In order to achieve this, wastewater has to enter the chamber tangentially and as such a centrifugal force will ensure that the grit is settled out. Vortex grit removal is installed in a channel with a narrower inlet and wider outlet. A motorized impeller is used to maintain the centrifugal force over a wider range of flow conditions. Grit is removed from the bottom of the tank either by means of grit pumps or airlift pumps to a grit classifier. Figure 3 shows an example of vortex grit chamber.

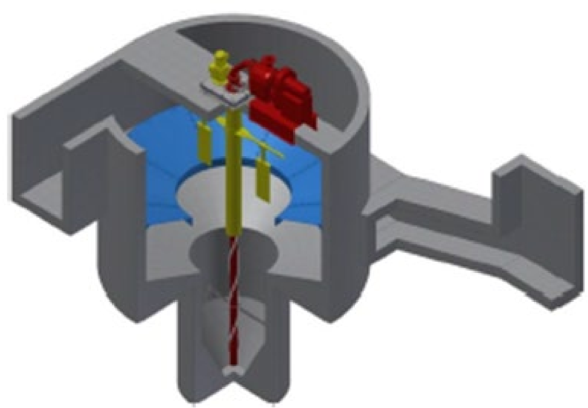

Figure 3 Typical Vortex Grit Chamber (Courtesy of Veolia)

A summary of advantages and disadvantages of aerated grit removal is provided in Table 8. Capital costs are comparable for both aerated grit removal and vortex grit removal systems.

Table 8 Summary of Advantages and Disadvantages of Vortex Grit Removal

Advantages	Disadvantages
• Proven technology • Smaller space requirements • Reduced odour potential • Lowest energy cost • There is no submerged bearings or parts requiring maintenance. Simplified conveyance • Small footprint • Low headloss	• The grit sump can be more prone to plugging in plants with extreme grit loads (i.e., combined sewer systems). Not expected to be a concern at SNF WWTP due to separate system and majority of flow pumped to plant.

The Region completed a preliminary assessment of the grit removal systems in all the Region's WWTP in November 2020. The purpose of the assessment was to investigate grit removal performance issues at the Fort Erie WWTP by comparing grit removal at the facility to performance of other plants owned by the Region. One of the outcomes of the study was that aerated grit systems had generally better removal rate of the smaller particles compared to vortex grit systems in the Region.

Aerated grit removal is recommended for the SNF WWTP headworks due to the following key advantages:

- On the Region's Approved Product and Equipment List.
- Similar capital costs to vortex grit removal.
- Maximizes grit removal.
- Exhibits consistent removal efficiency over a wide flow range.

3.2.3 Primary Treatment

Primary treatment is generally used to remove readily settleable solids and floating materials from the flow stream. The main objective of primary treatment is to reduce the load on the downstream biological treatment system and to provide a high energy value sludge that can increase energy production through digestion. Primary clarifiers typically remove 50 to 70 percent of TSS, 25 to 40 percent of BOD, and 10 to 20 percent TKN. By decreasing the biological load on the downstream biological treatment process, the aeration costs for the biological treatment process are also reduced. Primary clarification can also enhance biological phosphorous removal through the use of raw sludge fermenters to produce volatile fatty acids (VFAs).

The primary sludge produced in the primary clarifiers provides readily available biomass for digestion. Anaerobic digestion is the most applicable digestion process for primary sludge, which has a high methane gas yield and can produce clean energy while reducing the aeration requirement for the biological process. Also, primary sludge improves dewaterability of digested sludge when compared to waste activated sludge alone.

Due to the size of the facility, primary treatment is recommended for all secondary technology trains for the proposed SNF WWTP to:

- Reduce energy, as primary treatment provides substantial BOD and TSS removal, thus reducing the load on the downstream biological treatment processes.
- Maximize carbon capture
- Allow potential for energy recovery in solids management, as the primary sludge produced provides readily available biomass for anaerobic digestion.
- Provide flexibility for dual point chemical addition for improved phosphorus removal.

Two alternative technologies were considered for primary treatment:

- Alternative 1: Conventional Primary Clarifiers with Separate WAS Thickening
- Alternative 2: Conventional Primary Clarifiers with WAS Co-thickening

These alternatives will be based on conventional primary treatment, with provision made for chemical enhanced primary treatment (CEPT) for either option.

3.2.3.1 Conventional Primary Clarifiers with WAS Co-Thickening

For this alternative, sludge from the secondary treatment process would be co-thickened in the primary clarifiers and blended with primary sludge to digestion.

Co-thickening is mostly done in smaller facilities where they can reduce the number of mechanical equipment components. The practice of WAS co-thickening reduces primary clarifier capacity and commonly results in a more dilute sludge feed to the anaerobic digesters.

A summary of advantages and disadvantages of WAS co-thickening is provided in Table 9.

Table 9 Summary of Advantages and Disadvantages of WAS Co-thickening

Advantage	Disadvantage
• Simple operation and minimal monitoring required • Reduced equipment (no thickening unit, WAS Feed, polymer. TWAS feed)	• Significantly larger primary clarifiers. Increased odour potential with larger surface area. • This practice reduces primary clarifier capacity and results in a more dilute sludge feed to the anaerobic digesters. A larger sludge digestion and storage capacity would be required to handle the design flow. • Reduced site capacity.

3.2.3.2 Conventional Primary Clarifiers with Separate WAS Thickening

For this alternative, instead of WAS being co-thickened in primary clarifiers, sludge from the secondary treatment process would be thickened separately and blended with primary sludge to digestion.

With the provision of separate WAS thickening, the primary clarifiers can operate at a higher surface overflow rate (i.e., MECP Design Guideline peak day surface overflow rate (SOR) increased to 60-80 $m^3/m^2/d$ from 50-60 $m^3/m^2/d$), which will result in an increase in significantly smaller primary clarifiers as compared to the WAS co-thickening option. This option will also reduce the downstream anaerobic digestion and sludge haulage, through the reduction of sludge volume.

The primary disadvantage of separate WAS thickening is the need to operate mechanical thickening equipment.

A summary of advantages and disadvantages of Separate WAS thickening is provided in Table 10.

Table 10 Summary of Advantages and Disadvantages of Separate WAS Thickening

Advantage	Disadvantage
• Reduce downstream sludge digestion and storage capacity requirements, through feed sludge volume reduction • Less potential for primary treatment odour generation. • Lowest life-cycle cost with smaller primary clarifiers, anaerobic digesters and reduced sludge transport costs.	• New process to operate and maintain adding complexity to operations • Requires polymer addition to improve solids and liquid separation during WAS thickening process.

3.2.4 Secondary Treatment

The following technology alternatives were considered for secondary treatment for the SNF WWTP:

- Conventional Activated Sludge (CAS)
- Moving Bed Biofilm Reactor (MBBR)
- Biological Aerated Filter (BAF)
- Biological Nutrient Removal (BNR)
- Aerobic Granular Sludge (AGS)
- Membrane Aerated Biofilm Reactor (MABR)

3.2.4.1 Conventional Activated Sludge (CAS)

Conventional activated sludge (CAS) process consists of aeration tanks followed by secondary clarifiers. Microorganisms are maintained in suspension by aeration and mixed for effective contact with the influent (i.e., substrate) and dissolved oxygen (DO). Air is typically used as an oxygen source and it is common to supply it to the basin by diffusers; although other aeration systems can be used.

Effluent from the basin passes into the secondary clarifier where solids and microorganisms are settled out and returned to the aeration basin. Excess sludge is wasted from the system and generally further processed on-site. The returned solids are sent back to the head of the aeration basin to maintain the microbial concentration. This helps control solids retention time (SRT) independent of hydraulic retention time (HRT); thus, minimizing reactor volumes. Figure 4 shows a simplified process flow diagram for a typical CAS process.

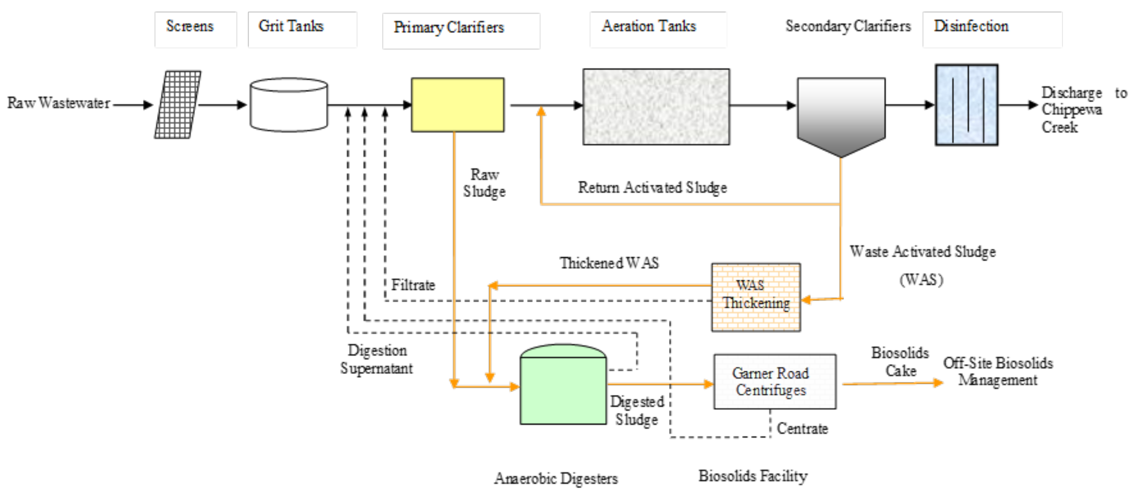

Figure 4 Typical CAS Process Flow Diagram

A summary of the advantages and disadvantages of the CAS process is provided in Table 11.

Table 11 Summary of Advantages and Disadvantages of CAS Technology

Advantages	Disadvantages
• Proven, robust treatment process with long history of application in similar climates • Low operational complexity • Flexible process with potential for advancing new technologies (i.e. aerobic granular sludge) • Lowest capital and life cycle cost (LCC)	• Process performance can be limited by sludge settleability • Relatively large footprint

The CAS has been widely used in the wastewater treatment facilities in Ontario and world-wide. It is the most common wastewater treatment technology.

3.2.4.2 Moving Bed Biofilm Reactor (MBBR)

The moving bed biofilm reactor (MBBR) process consists of an aeration basin filled with suspended media and a secondary clarifier. The process utilizes an inert carrier to support biomass growth and is essentially a high rate fixed film system.

This high rate process relies on the development of biofilm on small, lightweight, rigid plastic carrier media that fill the aeration tank and are kept in suspension by medium bubble diffusers and/or mixing. The plastic carrier elements have a high specific surface area for attached biomass growth, allowing for a more compact system compared to CAS. Screens are required within the tanks for media retention. When the media is used in conjunction with a CAS system (i.e., with mixed liquor recycle), the process is commonly termed Integrated Fixed Film Activated Sludge (IFAS).

The MBBR process does not require backwashing. Aeration tank effluent is clarified in a secondary clarifier, from which there is no recirculation of separated biomass. This results in a considerably lower solids loading rate on the secondary clarifiers relative to suspended growth systems. However, the settleability of the solids from the MBBR process is typically poorer than other CAS processes and can require polymer to aid in the settling of pin flocs.

Due to elimination of return activated sludge (RAS) cycle in an MBBR system, some operation costs may be saved as a result of the reduction in pumping requirements; however, these savings are offset by the increased aeration requirements due to the lower oxygen transfer efficiency of medium bubble aeration and higher DO operating set-point.

Figure 5 shows a simplified process flow diagram for a typical MBBR/IFAS process.

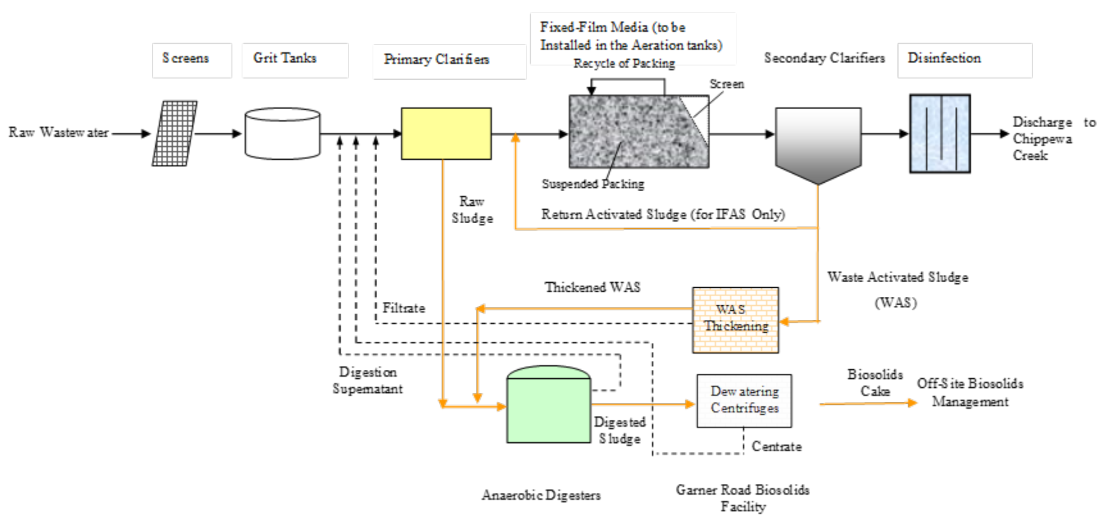

Figure 5 Typical MBBR/IFAS Process Flow Diagram

A summary of the advantages and disadvantages of the MBBR process is provided in Table 12.

Table 12 Summary of Advantages and Disadvantages of MBBR Technology

Advantages	Disadvantages
• Compatible with existing Niagara systems • Low operational complexity • Smaller footprint than CAS • Less susceptible to washout during peak wet weather flows	• High energy costs • Sludge can possess poor settling characteristics • Little control over effluent quality under varying environmental conditions

There has been limited full-scale applications of MBBR in Ontario. The Region is currently considering this technology for the upgrade of the existing Niagara Falls WWTP. The process has demonstrated the ability to achieve good removal of BOD_5 and nitrification even under the extreme winter climate.

3.2.4.3 Biological Aerated Filter (BAF)

Biological aerated filters (BAF) are high rate proprietary biological treatment process that uses an attached growth configuration to treat wastewater without requiring secondary clarification. The process consists of a biological reactor filled with 2 to 5 m media bed, which serves as both a filter and a surface area for biological activity.

The BAF process can be configured for carbon removal, nitrification, denitrification and chemical phosphorus removal. The most commonly used BAF process in Ontario is the BioStyr® process from Veolia. The BAF process is very much dependent on influent TSS, BOD and ammonia concentrations and generally requires a high-quality primary treatment. The process is periodically taken off-line for backwashing with BAF treated effluent.

The BAF process is compact (i.e., volumetric loading rates of up to an order of magnitude greater than with biotrickling filters), due to concentrated biomass and the combined function of biological treatment and solids separation within a single reactor. Their modular design is an advantage for future capacity upgrades for the case of the SNF WWTP.

Figure 6 shows a simplified process flow diagram for a typical BAF process.

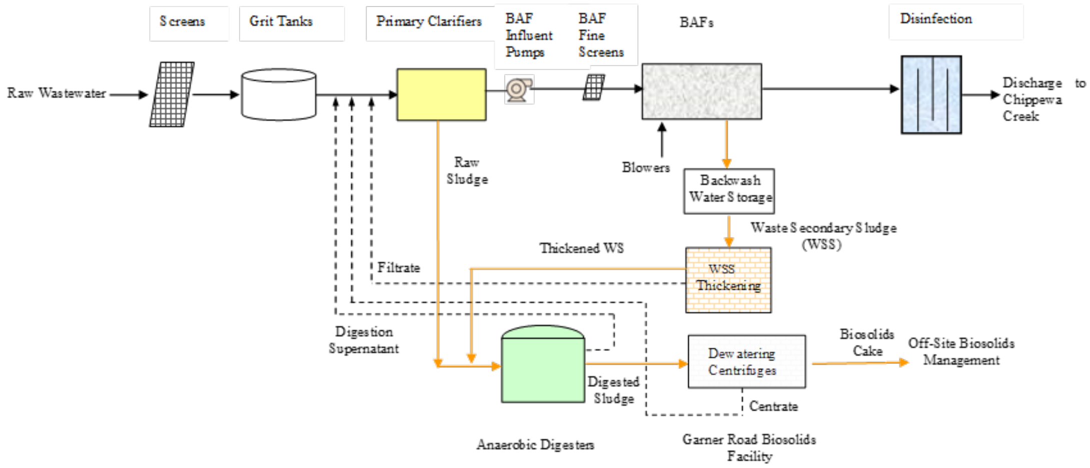

Figure 6 Typical BAF Process Flow Diagram

A summary of the advantages and disadvantages of the BAF technology is provided in Table 13.

Table 13 Summary of Advantages and Disadvantages of BAF Technology

Advantages	Disadvantages
• Smaller footprint than CAS • May be fully automated, reducing O&M costs • Flexible operation • Modular design – relatively simple capacity upgrades	• Complex mechanical and electrical control systems • Higher headloss due to inlet screening and filters which will likely require an intermediate pumping station assuming upstream primary clarifiers will be founded on competent soil • Higher ammonia concentration likely to require 2-stage treatment

There are some BAF applications in Ontario, including the Thunder Bay WWTP and plants that were recently upgraded from primary treatment to secondary treatment with limited footprint available and/or poor geotechnical conditions (i.e., rock excavation) such as the Owen Sound and Kingston WWTPs.

3.2.4.4 Biological Nutrient Removal (BNR)

Biological nutrient removal (BNR) processes are a modification of the CAS process to provide biological nitrogen and/or phosphorus removal. This is achieved through a three-stage process using Anaerobic/Anoxic/Aerobic to promote organisms that remove additional phosphorus biologically.

The inclusion of anaerobic zones, which are zones where dissolved oxygen and nitrate are absent, allows for the selection and growth of phosphorus-accumulating organisms (PAOs), which provide biological phosphorus removal. Effluent phosphorous concentrations produced are equivalent to other secondary treatment processes.

Biological phosphorus removal can reduce or eliminate the use of chemical for phosphorus removal. A disadvantage of the BNR plants is that they require larger bioreactor volume and footprint than plants designed only for nitrification and operation is less familiar to operations staff in Ontario.

Figure 7 shows a simplified process flow diagram for a typical BNR process.

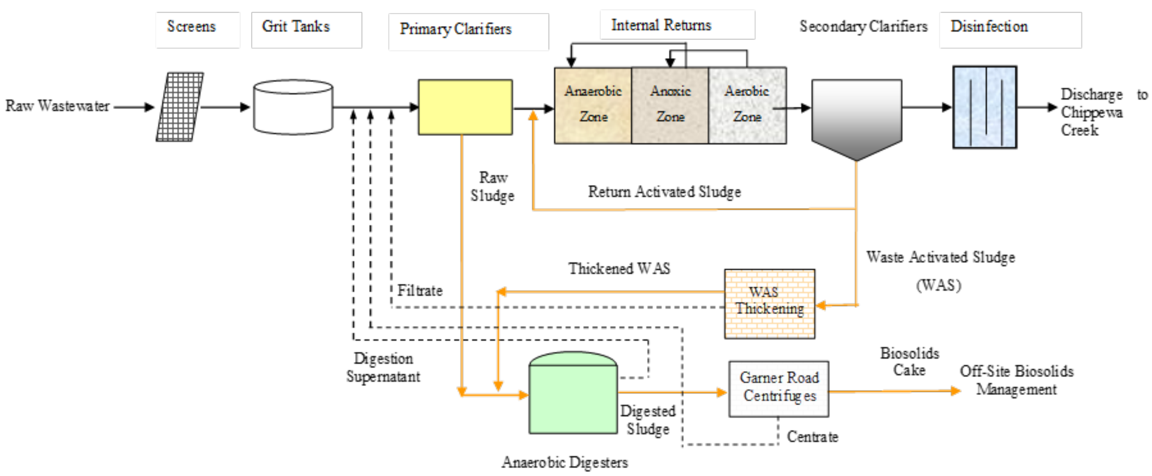

Figure 7 Typical BNR Process Flow Diagram

A summary of the advantages and disadvantages of the BNR technology is provided in Table 14.

Table 14 Summary of Advantages and Disadvantages of BNR Technology

Advantages	Disadvantages
• Low chemical consumption for phosphorus removal • Lowest biosolids volume generated • Lower chemical costs (reduced chemical for phosphorous removal)	• Higher land requirement (additional biological tanks for nutrient removal) compared to CAS • More complex operating requirements • May require chemical addition, specifically on recycle streams, to meet low TP effluent limits.

3.2.4.5 Granular Sludge / Ballasted Flocculation

Aerobic Granular Sludge (AGS) process is an advanced technology for biological wastewater treatment based on the sequencing batch reactor (SBR) system, using the advantages of the aerobic granular biomass. Bacteria grow in a natural way in compact granules instead of flocs, providing better settleability characteristics. There is on-going research to develop aerobic granular sludge technology for continuous flow activated sludge systems.

To develop the granules, aerobic, anoxic and aerobic phases of the SBR process occur within the same tank. Figure 8 shows the microscopic images of conventional activated sludge and aerobic granular sludge.

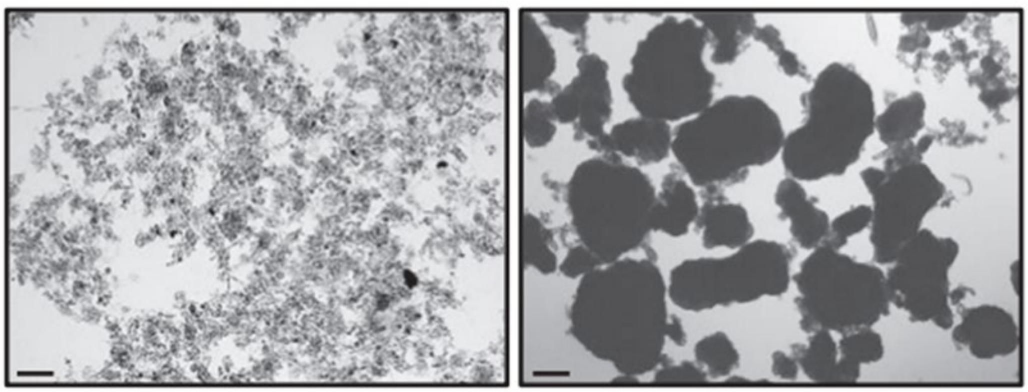

Figure 8 Microscopic Images CAS (left) and AGS (right)

Granular sludge process research and application has primarily used a SBR configuration (US EPA, 2013). A similar process has recently been developed for a continuous flow configuration (Caprariu, 2017) that requires the addition of an inert carrier media (ballast) for biomass attachment. As a result, the content of biomass in the aeration tanks and solids loading rate (SLR) to the clarifiers can be increased significantly. Plant re-rating through addition of AGS technology as it matures for continuous flow applications is possible in the future.

One vendor was reviewed in this memo which is based on MIMICS® (granules) and S:Select® (the process), represented locally by ETA.

The granules have diameter of less than 1 mm, which act as a colonization surface for the biomass. The MIMIC® granules are separated from WAS by hydrocyclones and recycled back to the aeration tank. The sheared biomass, as dilute WAS, with concentrations slightly higher than the aeration tank MLSS is conveyed to sludge treatment facilities. Figure 9 shows an example of S:Select® Hydrocyclones.

Figure 9 Select® Hydrocyclones and Recycle System (ETA)

The improvements to activated sludge resulting from this technology include:

- High settling velocity of sludge
- Small footprint requirement and provides high capacity increase (by factor of 2 to 3) as an add-on to conventional process.

The main drawback of this technology is risk of inert media being carried over in the effluent and/or sludge streams. However, there is significant on-going research to produce granules without the need for inert media.

A simplified process flow diagram of the granular sludge process would be the same as that of the CAS as shown in Figure 4. A summary of the advantages and disadvantages of the AGS technology is provided in Table 15.

Table 15 Summary of Advantages and Disadvantages of Granular Sludge Technology

Advantages	Disadvantages
• Smaller footprint • Simultaneous nitrification/denitrification and lower energy due to operating regime	• Developing technology – limited full-scale application in North America; but a number of plants using batch technology worldwide • May require pilot testing prior to full-scale implementation and MECP approval

The granular sludge technology is considered an emerging technology. Currently, there are over 30 full-scale installations in either construction or operation in other parts of world, but none operating in Ontario or Canada. As a result, MECP approval without site specific pilot testing will be challenging.

3.2.4.6 Membrane Aerated Biofilm Reactor (MABR)

The Membrane Aerated Biofilm Reactor (MABR) process employs a gas transfer membrane to deliver oxygen to a biofilm that grows on the surface of a membrane. The technology is being evaluated in some installations for its potential to increase existing treatment capacity by providing nitrification in a smaller tank volume than that required for conventional treatment. This effectively expands the capacity of the existing treatment plant, without the need to construct additional infrastructure. Figure 10 shows MABR operating principles.

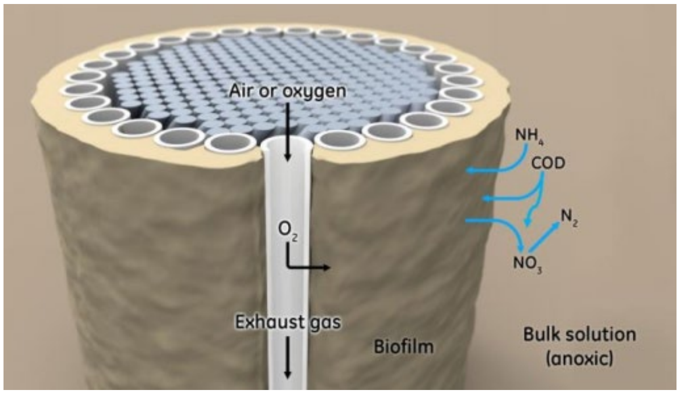

Figure 10 MABR Operation Principle (Courtesy of Suez)

A significant benefit of the MABR technology is the potential to reduce the energy consumption required for aeration by up to 30% compared to the current conventional treatment process. The significant energy savings for MABRs result from the delivery of oxygen at an efficiency up to four times greater than fine bubble aeration.

Nutrient removal is also enhanced for the MABR technology as the biomass inventory is increased by supplementing a suspended growth system with attached growth and enabling simultaneous nitrification and denitrification in the existing tank footprint.

A summary of the advantages and disadvantages of the MABR technology is provided in Table 16.

Table 16 Summary of Advantages and Disadvantages of MABR Technology

Advantages	Disadvantages
• Smaller Footprint • Reduced energy (very efficient, lower pressure oxygen transfer across membrane) • Simultaneous nitrification/denitrification (reduced effluent nitrate)	• Developing technology – limited full scale applications • Capital and long-term operating and maintenance (O&M) costa not well understood • MECP approvals may require pilot testing

The MABR technology is still considered an emerging technology with limited full-scale applications. In Ontario, the Hespeler WWTP in Waterloo and the North Toronto TP in Toronto are currently constructing full-scale installations of MABR technology.

3.2.5 Disinfection

The following alternatives were considered for secondary effluent disinfection:

- Alternative 1: Chlorination/dechlorination.
- Alternative 2: Ultraviolet (UV) disinfection.
- Alternative 3: Peracetic acid (PAA).

These alternatives were described as follows.

3.2.5.1 Chlorination/Dechlorination

This alternative would involve disinfection of the secondary effluent using chlorination followed by dechlorination.

Chlorine is added to inactivate pathogens and the residual chlorine is removed so that aquatic life in the receiving water are not impacted. The chemical disinfectant typically is supplied as liquid chlorine (sodium hypochlorite) or chlorine gas. Sodium hypochlorite is used at most Niagara Region WWTPs.

Dechlorination of the effluent is required to virtually eliminate chlorine residual in the receiver, which is toxic to aqua life. The most commonly used chemical for dechlorination at Niagara Region WWTPs is liquid sodium bisulphite.

One of the benefits of this process is that it can also be used to disinfect any lower quality wet weather bypass flows (i.e., secondary bypass).

Table 17 provides a summary of the advantages and disadvantages of the chlorination and dechlorination system.

Table 17 Summary of Advantages and Disadvantages of Chlorination/Dechlorination System

Advantages	Disadvantages
• Simple maintenance requirements • Familiar to Region staff • Ability to disinfect lower quality wastewaters • Cost comparable with UV system	• Health & safety risk with handling chemicals • Some traffic impacts for chemical delivery

Chlorination/dechlorination has been widely used across Ontario and is accepted by MECP.

3.2.5.2 UV Disinfection

This option would involve using ultraviolet (UV) irradiation to disinfect the secondary effluent (and any secondary treatment bypass flow - i.e. primary effluent).

UV disinfection process uses energy from mercury arc lamps to destroy or inactivate cells of bacteria and viruses. The UV disinfection process can be accomplished in either a closed vessel or open channel reactor, although the majority of wastewater installations are open channel. UV light can be produced by low-pressure or medium-pressure. Most newer installations are low-pressure using high-intensity lamps reducing energy and footprint requirements. The use of UV light as a disinfectant does not create any DBP formation and no in-stream chemicals are required to achieve primary disinfection.

The UV process has a relatively high electrical power consumption that can contribute to higher O&M costs. UV disinfection of lower quality wet weather bypass streams can require significantly more lamps and energy.

Table 18 provides a summary of the advantages and disadvantages of the UV disinfection system.

Table 18 Summary of Advantages and Disadvantages of UV Disinfection System

Advantages	Disadvantages
• Non-toxic effluent • Reduced chemical handling with associated risks	• High energy cost • Requires replacement and maintenance of lamps • Less effective at disinfecting lower quality wet weather bypass flow

The UV disinfection method has become common in municipal wastewater treatment over the past 30 years and is gaining more and more popularity in recent years. Some of the significant factors that have attributed to the growing popularity of UV radiation are:

- Increasing awareness of the impact of chlorine and chlorinated compounds on the environment; and
- Improvements in UV systems technology and equipment, resulting in a fewer lamps and improved efficiency.

3.2.5.3 Peracetic Acid (PAA)

Peracetic acid (PAA) is a very strong oxidizer and is commercially available as a liquid diluted with acetic acid, hydrogen peroxide and water. PAA has a broad spectrum of antimicrobial activity (effective at killing bacteria, fungus, and spores), as such it is a good secondary effluent disinfectant and eliminates the presence of residual chlorine. A contact time of 15 to 18 minutes is typically needed at average flow conditions and 8 to 10 minutes at peak flow. Even though

PAA is more environmentally friendly disinfectant, newer research showed that PAA still poses some risks and needs to be quenched before discharging to the receiving body.

Table 19 provides a summary of the advantages and disadvantages of the PAA disinfection system.

Table 19 Summary of Advantages and Disadvantages of PAA Disinfection System

Advantages	Disadvantages
• Eliminates the presence of residual chlorine • Lower potential of disinfection by-products formation compared to chlorine	• Higher cost when compared to chlorine and limited bulk availability • Lower efficiency against some viruses and parasite compared to chlorine • Increase in the effluent organic content, enhancing microbial regrowth • Relatively new technology

The PAA technology is considered a new technology Pilot studies/trials were conducted at in several Ontario WWTPs, including Ashbridges Bay WWTP, Hespeler WWTP and two WWTPs in the Niagara Region.

Currently in Ontario, there is no official MECP approval guidance for using PAA. As per communication with MECP, a DRAFT approval guidance is in process of internal review which requires site-specific piloting for each individual plant due to the limited full-scale application experience in Ontario.

3.2.6 WAS Thickening

Refer to Section 3.2.3 for WAS thickening options for SNF WWTP.

3.2.7 Digestion

With primary treatment, anaerobic digestion is recommended for sludge stabilization at the proposed SNF facility. As discussed in Section 3.2.3, the primary sludge produced from primary treatment is ideal for anaerobic digestion, as it can provide readily available biomass for digestion and has a higher energy production potential (i.e. higher methane gas yield) compared to an extended aeration plant that only generates WAS. Anaerobic digestion is a widely used sludge stabilization process including several wastewater treatment facilities in the Region. Anaerobic digestion involves decomposition of organic matter by microorganisms in the absence of oxygen at an elevated temperature for a period of approximately 15 days or more. Anaerobic digestion can typically achieve approximately 50 percent destruction of volatile solids (VS). Anaerobic digestion can be operated at both mesophilic (29-38 °C) and thermophilic (52 °C) temperatures; although all of the Region digestion facilities operate in the mesophilic range.

The anaerobic digestion process converts the sludge into biogas which is rich in methane, and leaves the resultant stabilize biosolids. The biosolids from mesophilic digestion are a Class B material that is compatible with the Region's current biosolids management program that includes a feedstock for fertilizer production (by a third party) and seasonal beneficial re-use on agricultural land. Most installations use the biogas to power boilers for process and plant heating needs. The biogas can also be used to generate electricity and heat (combined heat and power or CHP) or purified to renewable natural gas for injection to the utility grid.

Table 20 provides a summary of the advantages and disadvantages of anaerobic digestion.

Table 20 Summary of Advantages and Disadvantages of anaerobic digestion

Advantages	Disadvantages
• Increased volatile solids reduction when compared to conventional aerobic digestion; • Low life cycle costs, • Lower tank volume required compared to aerobic digestion; • Process generates methane, a renewable energy source; and • Higher volatile solids reduction.	• Potential for odor and foam formation; • Relatively high capital cost; • Supernatant has high ammonia concentrations, impacting liquid treatment process; and • Struvite formation potential.

For the SNF WWTP Phase 1 capacity (30 MLD), dewatering at the nearby Garner Road Biosolids facility is recommended. As the facility capacity continues to grow (to 60 MLD at Phase 2), opportunities can be reviewed for construction of a dedicated on-site dewatering and truck loading at the SNF WWTP should be considered.

3.3 Screening of Long List Technologies

The purpose of screening of long-list alternatives is to produce a short list of technology alternatives for more detailed evaluation.

3.3.1 Screening Criteria

In order to determine the most applicable, practical, and beneficial wastewater treatment technologies for the proposed SNF WWTP, a set of "must-meet" criteria were developed to screen each of the treatment technology alternatives. If any single criterion was not met for a given alternative, then it was not included in the short-list of options to be considered for the plant expansion. In other words, each alternative must meet all screening criteria. The screening criteria are presented in Table 21.

Table 21 "Must-Meet" Screening Criteria for Short-Listing Alternatives

Criterion	Description
Track Record to Meet Effluent Requirements	Demonstrated track record of ability to continuously meet and exceed the proposed treatment objectives
Scalability (two years in similar sized facility)	Demonstrated reliability of a successful two-year full-scale experience in similar sized facility.
Staging / phasing	Ability to expand to suit housing development's growth requirements
Capital and O&M Costs	Have a capital cost commensurate with the benefits provided

3.3.2 Screening of Technologies

A number of treatment technologies available for each treatment process were identified and described in Section 3.2. The long list of primary, secondary, disinfection, WAS thickening and digestion treatment technologies are described and screened against the specific screening criteria below from Table 22 to Table 27. The screening results are summarized in Table 28.

Table 22 Screening of Long List of Treatment Technologies for Preliminary Treatment

Preliminary Treatment: Technology	Track Record	Scalability	Staging / phasing	Cost	Carry Forward?	Rationale
Step Screens (6 mm)	Yes	Yes	Yes	Yes	Yes	• This technology is on the Region's Approved Equipment and Product list.
Aerated Grit Removal	Yes	Yes	Yes	No	Yes	• Higher operational cost than vortex grit removal; however better removal efficiency • This technology is selected due to ability to better protect downstream equipment
Vortex Grit Removal	Yes	Yes	Yes	Yes	No	• Not selected due to lower grit removal efficiency

Table 23 Screening of Long List of Treatment Technologies for Primary Treatment

Primary Treatment: Technology	Track Record	Scalability	Staging / phasing	Cost	Carry Forward?	Rationale
Conventional Primary Clarifiers with Co-thickening	Yes	Yes	Yes	No	Yes	• This option reduces primary clarifier capacity and treatment performance • Requires larger capacity of downstream sludge digestion and storage processes • Simplifies operation
Conventional Primary Clarifiers with Separate WAS Thickening	Yes	Yes	Yes	Yes	Yes (Future)	• Higher capital cost • This option minimizes footprint and therefore odour potential • More complex operation

Table 24 Screening of Long List of Treatment Technologies for Secondary Treatment

Secondary Treatment: Technology	Track Record	Scalability	Staging / phasing	Cost	Carry Forward?	Rationale
Conventional Activated Sludge (CAS)	Yes	Yes	Yes	Yes	Yes	• Proven, robust treatment process • Low operational complexity • Flexible process with potential for upgrade to other types of processes (e.g. BNR, AGS and MABR)
Moving Bed Biofilm Reactor (MBBR)	Yes	Yes	Yes	No	No	• High operating costs • More suited for intensification • Requires inlet and outlet screening
Biological Aerated Filter (BAF)	Yes	Yes	Yes	Yes	Yes	• Smaller footprint than CAS • May be fully automated, reducing O&M costs • Flexible operation • Modular design, easier for future upgrade
BNR	Yes	Yes	Yes	Yes	Yes	• Reduced chemical costs for phosphorus removal • Reduced aeration energy requirements
Aerobic Granular Sludge (AGS)	No	No	Yes	Yes	No	• Does not have demonstrated track record • Does not have two-year full-scale experience in similar size in Canada
Membrane Aerated Biofilm Reactor (MABR)	No	No	Yes	Yes	No	• Does not have demonstrated track record • Does not have two-year full-scale experience in similar size

Table 25 Screening of Long List of Treatment Technologies for Disinfection

Disinfection: Technology	Track Record	Scalability	Staging / phasing	Cost	Carry Forward?	Rationale
Chlorination/ Dechlorination	Yes	Yes	Yes	Yes	Yes	• Reliable • Cost effective
UV Disinfection	Yes	Yes	Yes	Yes	Yes	• No chlorine residual • Does not require chemicals handling, and • Has proven track record in Ontario at plants with similar size
Peracetic acid (PAA)	No	No	Yes	No	No	• Haven't demonstrated in full scale. • Not viable due to scale and cost prohibitive

Table 26 Screening of Long List of Treatment Technologies for WAS Thickenings

WAS Thickenings: Technology	Track Record	Scalability	Staging / phasing	Cost	Carry Forward?	Rationale
Separate WAS thickening	Yes	Yes	Yes	No	Yes	• This option reduces primary clarifier capacity and treatment performance • Requires larger capacity of downstream sludge digestion and storage processes • Overall higher capital costs
WAS Co-thickening	Yes	Yes	Yes	Yes	Yes	• This option minimizes footprint and therefore odour potential

Table 27 Screening of Long List of Treatment Technologies for Digestion

Digestion: Technology	Track Record	Scalability	Staging / phasing	Cost	Carry Forward?	Rationale
Anaerobic Digestion	Yes	Yes	Yes	Yes	Yes	• Not viable due to scale and implementation of upstream primary treatment process • Potential for energy recovery and utilization

3.4 Summary

Table 28 provides a summary of short-list treatment technologies developed.

Table 28 Summary of Short-listed Treatment Technologies

Unit Process	Short Listed Technologies
Preliminary Treatment	• Step Screen (6mm) • Aerated Grit Removal
Primary Treatment	• Conventional Primary Clarifier with WAS CO Thickening, with Provision of CEPT
Secondary Treatment	• Conventional Activated Sludge Process (CAS) • Biological Aerated Filters (BAF) • Biological Nutrient Removal
Disinfection	• Chlorination/Dechlorination • UV Disinfection
WAS Thickening	• Co-thickening in primary clarifier • Provision for separate WAS Thickening for Future
Digestion	• Anaerobic Digestion

Detailed evaluation will be conducted for the following unit processes in Section 4:

- Secondary treatment
- Disinfection

4 Evaluation of Short List Treatment Technologies

4.1 Overview

This section provides a summary of each short-listed alternative for secondary treatment (coupled with primary treatment) and disinfection processes, as part of the detailed evaluation:

- Brief description of each alternatives
- Preliminary process sizing for each alternative, including tankage volume and site area requirements, and
- High-level capital, operating and maintenance (O&M) and life cycle cost estimates

4.2 Description of Short-Listed Alternatives

4.2.1 Secondary Treatment

The following alternative technologies were considered for the construction of the proposed SNF WWTP, to accommodate the Phase 1 rated ADF capacity of 30 MLD:

- Alternative 1 – Conventional Activated Sludge (CAS)
- Alternative 2: Biological Aerated Filter (BAF)
- Alternative 3: Biological Nutrient Removal (BNR)

4.2.1.1 Alternative 1: Conventional Activated Sludge (CAS)

This option is based on constructing the new SNF WWTP with a conventional activated sludge (CAS) process. To achieve the required Phase 1 capacity of 30 MLD and to meet the future effluent quality requirements, the following work would be required for the CAS process:

- Construction of two plug flow aeration tanks, with a total volume of approximately 13,400 m^3. The aeration tanks will be up to 6.0-meter deep with the depth selected depending on geotechnical conditions.
- Construction of two secondary clarifiers with a total surface area of approximately 3,000 m^2, complete with WAS and RAS pumping.

A pre-anoxic zone will be is integrated into the aeration tanks. The anoxic zone provides the following advantages:

- Low capital cost modification (by adding a concrete baffle);
- Improved sludge settleability, by creating a high F:M zone; and
- Alkalinity recovery while still meeting effluent ammonia objectives.

If CAS is the preferred option for the SNF WWTP, additional modelling will be completed during the conceptual design to refine unit process sizing. Phosphorus would be removed through metal salt addition.

4.2.1.2 Alternative 2: Biological Aerated Filter (BAF)

This option is based on constructing the new SNF WWTP with a biological aerated filter (BAF) process. The BAF process eliminates the need for the secondary clarifiers but requires additional areas for effluent storage and backwash water storage tanks.

BAF processes can be configured for carbon removal, nitrification, denitrification and chemical phosphorus removal. For the purpose of this evaluation, the BioStyr® was considered. To provide BOD and ammonia removal, a two-stage configuration consisting of carbonaceous tanks (C-BAF) and nitrifying tanks (N-BAF) is recommended. The C-BAF tanks will operate in series with N-BAF tanks during average day flow conditions; but will operate in parallel with the N-BAF tanks during high wet weather flow conditions.

To achieve the required Phase 1 capacity of 30 MLD and to meet the future effluent quality requirements, the following work would be required for the BAF system:

- Construction of five (5) C-BAF tanks (4 duty, 1 standby), each 107 m^2 with 3.5 to 5.0 meters of media.
- Construction of ten (10) N-BAF tanks (9 duty, 1 standby), each 107 m^2 with 3.5 to 5.0 meters of media.
- Installation of primary effluent pumps to convey the primary effluent to the BAF tanks.
- Installation of screens upstream of the BAF tanks to protect BAF filters.
- Installation of a secondary effluent/backwash water storage tank and backwash pumps.

4.2.1.3 Alternative 3: Biological Nutrient Removal (BNR)

This option is based on constructing the new SNF WWTP with biological nitrogen and phosphorus removal.

To achieve the required Phase 1 capacity of 30 MLD and to meet the future effluent quality requirements, the following work would be required for the BNR system:

- Construction of two plug flow bioreactors, with a total volume of approximately 19,000 m^3 assuming approximately 40% unaerated volume. Each bioreactor will be baffled into three separate zones arranged as follows: anaerobic, anoxic and aerobic. Recycle pumps will be provided within the bioreactors to allow for internal mixed liquor recycling from the aerobic zone to the anoxic zone. If BNR is the preferred alternative, additional modelling will be completed during conceptual design to refine the sizing and configuration of the tankage.

- Similar to Alternative 1-CAS, construction of two secondary clarifiers with a total surface area of 3,000 m^2, complete with WAS and RAS pumping.

It is noted that BNR technology will require significantly more infrastructure construction (e.g., creation of anaerobic/anoxic selector zones) and larger bioreactor volume, compared to Alternative 1 – CAS.

4.2.1.4 Cost Comparison

A 20-year net present value (NPV) life-cycle cost analysis was conducted for secondary treatment alternatives, including estimated capital, and operating and maintenance (O&M) costs, as follows:

- Conceptual capital costs generally included construction costs of new infrastructure /equipment such as bioreactors (i.e. aeration tanks, BAF tanks or BNR reactors) and secondary clarifiers.
- Operating costs included items to allow comparison between options, including energy, chemicals, equipment maintenance and labour. The costs do not include items that are common among options.
- Life cycle costs were calculated based on a 20-year life expectancy, with 2% inflation rate and 4% interest rate.

Table 29 presents a summary of cost comparison for the secondary treatment alternatives. Additional cost estimate details are provided in Appendix A.

It is important to note that these costs have been developed for comparison of alternatives. The overall design approach and cost will be refined for the preferred alternative during the conceptual design phase.

Table 29 Cost Estimates for Secondary Treatment Alternatives

Parameters	Alternative 1 -CAS	Alternative 2 -BAF	Alternative 3 -BNR
Capital Cost (1)	$55,986,000	$51,310,000	$71,610,000
Annual O&M Cost	$1,444,000	$1,838,000	$1,319,000
NPV 20-Year Life-Cycle Cost (2)	$78,610,000	$80,490,000	$91,880,000

Notes:

1. All costs are conceptual level opinions of probable costs in 2020 dollars and are accurate to +/-50%.
2. Based on a 2% inflation rate and 4% interest rate.

4.2.2 Effluent Disinfection

4.2.2.1 Chlorination/Dechlorination

This option is based on construction of chlorination/dechlorination facility downstream of the secondary clarifiers for secondary effluent disinfection.

To achieve the required Phase 1 capacity of 30 MLD and to meet the future effluent quality requirements, the following work would be required for the chlorination and dechlorination system:

- Construction of two (2) baffled chlorine contact tanks (each 15.0 m x 14.7 m x 2.5 m depth), with a total volume of 1,100 m^3. This will provide a 15 minutes chlorination contact time at the design peak hourly flows of 106 MLD (MECP, 2008).
- Construction of one (1) dechlorination tank, with a total volume of 75 m^3. This will provide a 60 seconds dechlorination contact time at the design peak hourly flows of 106 MLD.

4.2.2.2 UV Disinfection

This option is based on constructing a new UV disinfection facility downstream of the secondary clarifiers for disinfection of secondary effluent. For the purpose of this conceptual evaluation, a Trojan UV 3000 plus configuration with low-pressure high-intensity lamps was considered.

To achieve the required Phase 1 capacity of 30 MLD and to meet the future effluent quality requirements, the following work would be required for the UV disinfection facility:

- Construction of a UV disinfection building with a size of approximately 15 m x 10 m, which will allow for future building expansion for Phase 2 capacity.
- Installation of UV equipment within the UV building. Initially a total of two UV channel (1 duty, 1 standby) will be constructed for Phase 1, with two banks per channel.

4.2.2.3 Cost Comparison

A 20-year net present value (NPV) life-cycle cost analysis was conducted for effluent disinfection alternatives, including capital, and operating and maintenance (O&M) costs, as follows:

- Conceptual capital costs generally included construction costs of new infrastructure /equipment such as chlorine contact tanks, dichlorination contact tank, and/or UV disinfection facility.
- Operating costs included items to allow comparison between options, including energy, chemicals, equipment maintenance and labour. The costs do not include items that are common among options.
- Life cycle costs were calculated based on a 20-year life expectancy, with 2% inflation rate and 4% interest rate.

Table 30 presents a summary of cost comparison for the secondary treatment alternatives. Additional cost estimate details are provided in Appendix A.

It is important to note that these costs have been developed for comparison of alternatives. The overall design approach and cost will be refined for the preferred alternative during the conceptual design phase.

Table 30 Cost Estimates for Effluent Disinfection Alternatives

Parameters	Alternative 1 – Chlorination/Dichlorination System	Alternative 2 – UV System
Capital Cost (1)	$3,970,000	$4,860,000
Annual O&M Cost	$34,000	$69,000
NPV 20-Year Life-Cycle Cost (2)	$4,450,000	$5,899,000

Notes:

1. All costs are conceptual level opinions of probable costs in 2020 dollars and are accurate to +/- 50%.
2. Based on a 2% inflation rate and 4% interest rate.

4.3 Evaluation of Alternatives

4.3.1 Evaluation Methodology and Criteria

A decision-making model centered on a multi-criteria analysis (MCA) for the evaluation of short-listed sludge management alternatives. The MCA provides a structured approach to determine overall benefits among alternative options, where the options accomplish several objectives. This evaluation methodology requires specification of desirable objectives and identification of corresponding indicators, which are then used to measure/assess the ability of each alternative option to meet a specific objective.

The evaluation approach follows a typical evaluation of impacts to a wide range of criteria that include natural, socio/cultural, financial and technical environments, as well as legal/jurisdictional and technical factors (i.e., a triple bottom-line type of analysis). The decision-making criteria and rationale are summarized from Table 31 to Table 35.

Table 31 Decision-Making Criteria and Rationale for the Environmental Criteria (25%)

Sub Criteria	Criteria Rationale / Indicators
Potential Impact on Environmentally Sensitive Features	• Impact to environmentally sensitive features (e.g. Provincially Significant Wetlands (PSW), Environmental Sensitive Areas (ESA), Environmental Consideration Areas (ECA), Areas of Natural and Scientific Interest (ANSI), significant woodlots, creeks and other designated natural areas as per Official Plans (City of Niagara Falls or Niagara Region) and Niagara Peninsula Conservation Authority or Niagara Parks regulated areas • Maximizes natural buffer
Impact to Species at Risk	• Impact to Species at Risk and sensitive aquatic habitats (e.g. proximity to vulnerable/threatened/endangered or locally/regionally rare amphibians, wildlife or fish) • Impacts on sensitive terrestrial flora and fauna habitats
Potential Effects to Water Features/ Resources	• Impact on surface water levels (short or long-term) • Impact to crossing of floodplains and meander belts (e.g. potential flooding and erosion risk) • Impact on water quality, including nearby water sources and surface/groundwater
Receiving Waterbody	• Impact on effluent criteria and outfall considerations • Impact to health of receiving waterbody • Minimizes chemical components for treatment • Ability to meet regulatory requirements • Impact on recreational uses • Ability to protect existing water uses
Impact on System Overflows	• Maximize opportunities to reduce overflows • Ability to alleviate the existing system and strain on Stanley Avenue Wastewater Treatment Plant and related Sewage Pumping Stations
Physical Environmental Considerations (Geology, Hydrogeology, Soil/Land Contamination)	• Minimizes environmental crossings • Minimizes time required for contamination review/investigation/ remediation • Subsurface soils and rock characteristics, groundwater levels and water table levels • Level of short or long-term anticipated groundwater impacts (e.g. drilling through water table) • Investigation of potential Hydrogen Sulphide (H2S)

Sub Criteria	Criteria Rationale / Indicators
Climate Change	• Impact on long-term planning • Level of adaptability/resilience • Flexibility in operation and treatment needs • Minimize impact from climate or contributing to climate conditions
Environmental Risk	• Potential environmental risk during construction and/or operation • Potential for non-mitigatable impact • Potential for lack of success of the overflow strategy

Table 32 Decision-Making Criteria and Rationale for the Social/Cultural Criteria (25%)

Sub Criteria	Criteria Rationale / Indicators
Community Concerns for Residents/Local Businesses/ Traffic	• Reduces public health and safety concerns • Impact on recreational amenities • Existing and future employment and population areas • Impact of Wastewater Treatment Plant aesthetics • Impact on surrounding properties or public spaces • Impact on travel time during construction • Impact of temporary local disruption to road and public transit traffic • Ability to improve local aesthetics • Nature of adverse effects on roadway • Coordination with planned road work improvements
Impact on Indigenous Communities	• Impact during construction and operation • Impact on short and long-term planning
Impacts on Archaeological/ Cultural Heritage Features	• Impacts on nearby agricultural lands • Likelihood for impact to heritage homes/properties/landscape • Presence of known archaeological resources/sites, potential impacts on them and ability to mitigate • Number of known archeological sites affected

Sub Criteria	Criteria Rationale / Indicators
Air Quality and Odour Impact	• Impact to surrounding land users • Impact on life cycle air quality associated with overall servicing strategy • Impact on odour from operation of Sewage Pumping Stations, forcemain and/or sanitary sewers • Impact of H2S to create odorous environment • Impact of air quality surrounding the site or official regulations • Incorporation of treatment technologies
Noise, Vibration and Dust Impact	• Impact of factors: noise, vibration, dust (potential impacts - major, moderate, minor) associated with the impact factors
Compatibility with Current/ Planned Land Uses	• Suitability of land use designation
Overall Socio/Cultural Risk	• Cultural heritage or archaeological delays • Impact to community during construction and operation (odour, noise, etc.)
Community Concerns for Residents/Local Businesses/ Traffic	• Reduces public health and safety concerns • Impact on recreational amenities • Existing and future employment and population areas • Impact of Wastewater Treatment Plant aesthetics • Impact on surrounding properties or public spaces • Impact on travel time during construction • Impact of temporary local disruption to road and public transit traffic • Ability to improve local aesthetics • Nature of adverse effects on roadway • Coordination with planned road work improvements

Table 33 Decision-Making Criteria and Rationale for the Legal/Jurisdictional Criteria (10%)

Sub Criteria	Criteria Rationale / Indicators
Approvals/ Coordination Land Use Suitability	• Potential conflicts or conformity with City of Niagara Falls or Niagara Region Official Plan policies, including Secondary Plans, Master Servicing Plans, and Niagara Peninsula Conservation Authority or Niagara Parks regulations • Compliance with federal, provincial and local plans • Effluent criteria and outfall considerations • Minimize need for environmental approvals for removal of environmental features • Impact to aquatic or natural environments • Reduction of noise and odour impacts • Minimizes jurisdictional requirements (maximize infrastructure within existing regional road right of ways and minimize impact to Ontario Power Generation maintenance/operations) • Compatibility with existing future land use designations • Proximity to physical features (i.e. waterbodies/ highways/railways/residential/recreational)

Land Acquisition	• Land requirement issues and agency concerns that may arise related to project routes, siting and land acquisition
	• Site compatibility
	• Degree of complexity relating to:
	    • Availability of land
	    • Current designated land use
	    • Current ownership
	    • Property acquisition and easement requirements

Worker Safety and Operability	• Accessibility for operation and maintenance
Overall Legal / Jurisdictional Risk	• Complexity of land acquisition/ownership • Complexity of approvals/coordination

Table 34 Decision-Making Criteria and Rationale for the Technical Criteria (20%)

Sub Criteria	Criteria Rationale / Indicators
Technical Collection	• Opportunity to remove overflows • Opportunity to remove Sewage Pumping Stations • Flexibility for future servicing • Feasibility of costing • Ability to accommodate Thorold South • Ability to attenuate peak flows • Maximizes gravity
Technical Treatment	• Secondary or tertiary treatment requirements • Effluent Discharge requirements (present/future) • Disinfection options • Flow forecasts • Flexibility to incorporate treatment technologies • Potential for H2S gas during construction • Odour and noise remediation requirements
Technical Outfall	• Location and crossings • Impact on constructability • Ease of accessibility • Impact on outfall slope/depth/length • Impact on soil/groundwater/vegetation • Impact on receiving waterbody
Compatibility/ Impacts to Existing and Future Infrastructure	• Flexibility for future expansion, upgrades or connections • Ability to maximize use of existing infrastructure • Coordination opportunities with planned infrastructure improvements • Integration or Impact with existing utilities and other infrastructure and ability to maintain utilities and infrastructure in service • Utility easements within or in close proximity • Potential for infrastructure to impact recent or planned investments • Minimizes watercourse/highway/railway crossings

Sub Criteria	Criteria Rationale / Indicators
	• Construction in areas with limited access • Proximity and/or conflict with existing infrastructure • Accessibility and safety • Ability to maintain existing services during and following construction • Operational flexibility • Ability to meet future servicing needs for new growth and post 2041 projections • Flexibility with future servicing requirements • Maximize service area
Biosolids Strategy	• Close proximity to existing Biosolids Plant • Easy truck access to Biosolids • Ability to pump to Biosolids • Ability to minimize infrastructure needs
System Security and Level of Service	• Ability to maintain or enhance operational security • Ability to maintain or enhance service standard for the customer
Traffic Management	• Anticipated degree of construction truck traffic management issues during construction • Anticipated level of truck traffic during typical operations and maintenance
Operation & Maintenance	• Minimizes long-term operation and maintenance requirements • Ease of access to operate and maintain • Provision of emergency access • Deterioration (condensation, salt and H_2S)
Overall Technical Risk	• Impact on growth (capacity risk) • Overdesigning and stranding capacity (capacity risk) • Treatment technology risk • Construction risk • Schedule/timing risk

Table 35 Decision-Making Criteria and Rationale for the Financial Criteria (20%)

Sub Criteria	Criteria Rational / Indicators
Capital Cost	• Total capital (construction) cost for new infrastructure and/or upgrades for overall servicing strategy • Cost of required/needed property acquisition/easements
Lifecycle Cost (Operation, Resourcing, and Maintenance and Servicing)	• Minimizes operation & maintenance costs • Cost of operation and maintaining the infrastructure • Impact to regional resources • Ease of access to maintain • Provision of emergency access • Minimize total lifecycle cost (combination of capital, property acquisition, operation & maintenance, etc.) • Ability to decommission existing Sewage Pumping Stations • Minimize wastewater infrastructure footprint to reduce impact on climate
Cash Flow/Phasing of Costs	• Impact to cash management • Futureproof costing impact (i.e. potential for decisions or treatment options to become obsolete and difficult to replace in the future) • Phasing of costs and impact to DCs and rates
Funding Opportunities	• Developmental Charges • Grants (Federal, Provincial)
Overall Financial Risk	• Financial risk during construction (cost increase/ uncertainty) • Complexity of solution • Scope increase • Potential impact of unforeseen costs (capital/operations, etc.)

Based on the evaluation methodology and criteria described above, an evaluation matrix has been prepared describing the specific advantages and disadvantages that each alternative option offers for each criterion under consideration. Within the evaluation matrix, symbolic scores are assigned as follows to allow the relative ranking of each alternative:

● Lowest impact (meets criteria very well)
◕
◑
◔
○ Highest impact (meets criteria very poorly)

The scores are based on benefits, risk of impacts and mitigation requirements to minimize impacts for each specific alternative with respect to each criterion. The score for each criterion will be assigned and a preferred alternative with the most positive and least negative impacts will be selected.

4.3.2 Evaluation of Treatment Alternatives

The following wastewater treatment technology alternatives were evaluated for secondary treatment and effluent disinfection processes, based on the evaluation criteria in Table 26.

4.3.2.1 Secondary Treatment

- Alternative 1: CAS
- Alternative 2: BAF
- Alternative 3: BNR

4.3.2.2 Effluent Disinfection

- Alternative 1: Chlorination/Dechlorination
- Alternative 2: UV Disinfection

Detailed evaluation matrix of scoring and rationale for secondary treatment and effluent disinfection processes can be provided in Appendix B.

4.3.3 Summary

Based on the evaluation, the following are the preferred wastewater treatment technology alternatives for secondary treatment and effluent disinfection processes:

4.3.3.1 Secondary Treatment

CAS is the preferred secondary treatment alternative, due to the following key benefits:

- Proven technology
- Second lowest and life-cycle cost
- Easy operation and maintenance
- Ability to incorporate new technologies in the future

4.3.3.2 Effluent Disinfection

chlorination/dechlorination is the preferred effluent disinfection alternative, due to the following key benefits:

- Simple maintenance requirements
- Familiar to Region staff
- Ability to disinfect lower quality wastewaters
- Lowest capital and life cycle costs

5 Summary and Recommendations

In this Technical Memorandum, alternative technologies for each unit treatment process for both liquid and solids trains have been identified and evaluated for the new SNF WWTP. Table 36 provides a summary of the recommendations.

Table 36 Summary of Treatment Technology Recommendations

Unit Process	Recommended Technologies
Preliminary Treatment	• Step Screen (6mm) • Aerated Grit Removal
Primary Treatment	• Conventional Primary Clarifier with WAS CO Thickening, with Provision of CEPT
Secondary Treatment	• Conventional Activated Sludge Process (CAS)
Disinfection	• Chlorination/Dechlorination
WAS Thickening	• Co-thickening in primary clarifier • Provision for separate WAS Thickening for Future
Digestion	• Anaerobic Digestion

6 References

- GM Blue Plan (2017). Niagara Region 2017 Water and Wastewater Master Servicing Plan Update.
- Metcalf & Eddy, Inc. (2003), Wastewater Engineering: Treatment and Re-Use 4th Ed.", New York, New York, U.S.A.
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- Water Environmental Federation (WEF, 2017). Design of Water Resources Recovery Facilities MOP 8, Fifth Edition.

A

Appendix A
Cost Estimates

Project Title:	**SNF WWTP Class EA and Conceptual Design - TM No. 2 - Technology Review**		
Client:	Regional Municipality of Niagara		
Project No.:	T001140A		
Task:	Opinion of Capital Cost, O&M Cost and Life Cycle Cost Estimates		
Revision No. :	1	**Revision Date:**	09-Jul-20

Alternative 1 - Conventional Activated Sludge (CAS)

Opinion of Probable Cost

Item	Quantity	Unit	Unit Cost	Installatiion Factor	Subtotal Cost	Comments
Secondary Treatment System						
Aeration Tanks	13,400	m^3	$1,800	1	$24,120,000	Total AT Volume: 13,400 m^3; Incl. Blower Bldg, and aeration equipment
Secondary Clarifiers	3,000	m^2	$4,000	1	$12,000,000	Total SC Surface Area: 3,000 m^2; includes RAS/WAS Pumping
Subtotal Capital Cost					**$36,120,000**	
Engineering (15%)					$5,418,000	15%
Contigency Cost (5%)					$1,806,000	5%
Estimating Allowance (20%)					$7,224,000	20%
General Contractor's Overhead & Profit, Mob.,bond (15%)					$5,418,000	15%
Total Project Capital Cost (Excluding HST)					**$55,986,000**	

Annual Operation & Maintenance Cost (at Phase 1 Rated Capacity of 30 MLD)

Description	Annual Cost	Comments
Energy	$788,400	$0.15/kWh, two duty 300 kW blowers, 24 hrs/d
Chemical Consumption	$60,000	for phosphorus removal
Equipment Maintenance/Replacement	$450,000	3% of equipment cost
Labor	$146,000	$50/hr; 8 hrs/d
Total Annual O&M Cost	**$1,444,000**	

Life Cycle Cost

Economic Factors

Interest rate (%) 4.0%

Inflation rate (%) 2.0%

Cost in Year n = Cost in Current Year x (1+inflation Rate)^(Year n - Current Year)

Present Value = Cost /((1+Interest Rate)^(Year n - Current Year))

NPV					
Year	**Capital Cost**	**NPV Capital Cost**	**Operating Cost**	**NPV Operating Cost**	**Capital and Operating NPV**
	(2020$)		(2020$)		
2020	$0				
2021	$55,986,000	$54,909,346	$1,444,000	$1,416,231	$56,325,577
2022	$0	$0	$1,444,000	$1,388,996	$1,388,996
2023	$0	$0	$1,444,000	$1,362,284	$1,362,284
2024	$0	$0	$1,444,000	$1,336,086	$1,336,086
2025	$0	$0	$1,444,000	$1,310,392	$1,310,392
2026	$0	$0	$1,444,000	$1,285,193	$1,285,193
2027	$0	$0	$1,444,000	$1,260,477	$1,260,477
2028	$0	$0	$1,444,000	$1,236,237	$1,236,237
2029	$0	$0	$1,444,000	$1,212,464	$1,212,464
2030	$0	$0	$1,444,000	$1,189,147	$1,189,147
2031	$0	$0	$1,444,000	$1,166,279	$1,166,279
2032	$0	$0	$1,444,000	$1,143,850	$1,143,850
2033	$0	$0	$1,444,000	$1,121,853	$1,121,853
2034	$0	$0	$1,444,000	$1,100,279	$1,100,279
2035	$0	$0	$1,444,000	$1,079,120	$1,079,120
2036	$0	$0	$1,444,000	$1,058,368	$1,058,368
2037	$0	$0	$1,444,000	$1,038,014	$1,038,014
2038	$0	$0	$1,444,000	$1,018,053	$1,018,053
2039	$0	$0	$1,444,000	$998,475	$998,475
2040	$0	$0	$1,444,000	$979,273	$979,273
Sub-Total NPV value =		$54,909,346		$23,701,070	
Total NPV value =			**$78,610,000**		**$78,610,000**

Project Title:	**SNF WWTP Class EA and Conceptual Design - TM No. 2 - Technology Review**
Client:	Regional Municipality of Niagara
Project No.:	T001140A
Task:	Opinion of Capital Cost, O&M Cost and Life Cycle Cost Estimates
Revision No. :	1 — **Revision Date:** 09-Jul-20

Alternative 2 - Biological Aerated Filters (BAF)

Opinion of Probable Cost

Item	Quantity	Unit	Unit Cost	Installatiion Factor	Subtotal Cost	Comments
Secondary Treatment System						
Civil Works (BAF Tankage and Gallery)	16,000	m^3	$600	1	$9,600,000	15 filters plus gallery
BAF Equipment Supply and Installation	1	EA	$8,000,000	2	$16,000,000	
Primary Effluent PS	1	EA	$5,500,000	1	$5,500,000	
BAF Screening	1	EA	$2,000,000	1	$2,000,000	
Subtotal Capital Cost					**$33,100,000**	
Engineering (15%)					$4,965,000	15%
Contigency Cost (5%)					$1,655,000	5%
Estimating Allowance (20%)					$6,620,000	20%
General Contractor's Overhead & Profit, Mob.,bond (15%)					$4,965,000	15%
Total Project Capital Cost (Excluding HST)					**$51,310,000**	

Annual Operation & Maintenance Cost (at Phase 1 Rated Capacity of 30 MLD)

Description	Annual Cost	Comments
Energy	$1,169,460	$0.15/kWh. Two duty 300 kW blowers and two 45 kW BAF influent pumps, two 50 kW backwash pumps, 50 kW interstage pumps, 24 hrs/d
Chemical Consumption	$60,000	for phosphorus removal
Equipment Maintenance/Replacement	$390,000	3% of equipment cost
Labor	$219,000	$50/hr; 12 hrs/d
Total Annual O&M Cost	**$1,838,000**	

Life Cycle Cost

Economic Factors

Interest rate (%)	4.0%
Inflation rate (%)	2.0%

Cost in Year n = Cost in Current Year x (1+inflation Rate)^(Year n - Current Year)
Present Value = Cost /((1+Interest Rate)^(Year n - Current Year))

NPV					
Year	Capital Cost	NPV Capital Cost	Operating Cost	NPV Operating Cost	Capital and Operating NPV
	(2020$)		(2020$)		
2020	$0				
2021	$51,310,000	$50,323,269	$1,838,000	$1,802,654	$52,125,923
2022	$0	$0	$1,838,000	$1,767,987	$1,767,987
2023	$0	$0	$1,838,000	$1,733,988	$1,733,988
2024	$0	$0	$1,838,000	$1,700,642	$1,700,642
2025	$0	$0	$1,838,000	$1,667,937	$1,667,937
2026	$0	$0	$1,838,000	$1,635,861	$1,635,861
2027	$0	$0	$1,838,000	$1,604,403	$1,604,403
2028	$0	$0	$1,838,000	$1,573,549	$1,573,549
2029	$0	$0	$1,838,000	$1,543,288	$1,543,288
2030	$0	$0	$1,838,000	$1,513,609	$1,513,609
2031	$0	$0	$1,838,000	$1,484,502	$1,484,502
2032	$0	$0	$1,838,000	$1,455,953	$1,455,953
2033	$0	$0	$1,838,000	$1,427,954	$1,427,954
2034	$0	$0	$1,838,000	$1,400,494	$1,400,494
2035	$0	$0	$1,838,000	$1,373,561	$1,373,561
2036	$0	$0	$1,838,000	$1,347,146	$1,347,146
2037	$0	$0	$1,838,000	$1,321,240	$1,321,240
2038	$0	$0	$1,838,000	$1,295,831	$1,295,831
2039	$0	$0	$1,838,000	$1,270,912	$1,270,912
2040	$0	$0	$1,838,000	$1,246,471	$1,246,471
	Sub-Total NPV value =	$50,323,269		$30,167,982	
	Total NPV value =		**$80,490,000**		**$80,490,000**

Project Title:	**SNF WWTP Class EA and Conceptual Design - TM No. 2 - Technology Review**		
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Alternative 3 - Biological Nutrient Removal (BNR)

Opinion of Probable Cost

Item	Quantity	Unit	Unit Cost	Installatiion Factor	Subtotal Cost	Comments
Secondary Treatment System						
Aeration Tanks	19,000	m^3	$1,800	1	$34,200,000	Total Volume: 19,000 m^3 Includes blower building, aeration, mixin and recirc equipment
Secondary Clarifiers	3,000	m^2	$4,000	1	$12,000,000	Total Surface Area: 3,000 m2
Subtotal Capital Cost					**$46,200,000**	
Engineering (15%)					$6,930,000	15%
Contigency Cost (5%)					$2,310,000	5%
Estimating Allowance (20%)					$9,240,000	20%
General Contractor's Overhead & Profit, Mob.,bond (15%)					$6,930,000	15%
Total Project Capital Cost (Excluding HST)					**$71,610,000**	

Annual Operation & Maintenance Cost (at Phase 1 Rated Capacity of 30 MLD)

Description	Annual Cost	Comments
Energy (Aeration)	$591,300	$0.15/kWh, two duty 225 kW blowers, 24 hrs/d
Energy (Mixing/Recirculation)	$85,410	5 W/m3 unaerated volume, 0.3 kW per 1000 m3/d recirculation; 3 Q recirculation
Chemical Consumption	$0	for phosphorus removal
Equipment Maintenance/Replacement	$540,000	3% of equipment cost
Labor	$219,000	$50/hr; 12 hrs/d
Sludge Management Credit	($116,800)	0.8 dT/d (approx. 10% reduction), $400/dT
Total Annual O&M Cost	**$1,319,000**	

Life Cycle Cost

Economic Factors

Interest rate (%) 4.0%
Inflation rate (%) 2.0%

Cost in Year n = Cost in Current Year x (1+inflation Rate)^(Year n - Current Year)
Present Value = Cost /((1+Interest Rate)^(Year n - Current Year))

NPV					
Year	**Capital Cost**	**NPV Capital Cost**	**Operating Cost**	**NPV Operating Cost**	**Capital and Operating NPV**
	(2020$)		(2020$)		
2020	$0				
2021	$71,610,000	$70,232,885	$1,319,000	$1,293,635	$71,526,519
2022	$0	$0	$1,319,000	$1,268,757	$1,268,757
2023	$0	$0	$1,319,000	$1,244,358	$1,244,358
2024	$0	$0	$1,319,000	$1,220,428	$1,220,428
2025	$0	$0	$1,319,000	$1,196,958	$1,196,958
2026	$0	$0	$1,319,000	$1,173,940	$1,173,940
2027	$0	$0	$1,319,000	$1,151,364	$1,151,364
2028	$0	$0	$1,319,000	$1,129,222	$1,129,222
2029	$0	$0	$1,319,000	$1,107,507	$1,107,507
2030	$0	$0	$1,319,000	$1,086,208	$1,086,208
2031	$0	$0	$1,319,000	$1,065,320	$1,065,320
2032	$0	$0	$1,319,000	$1,044,833	$1,044,833
2033	$0	$0	$1,319,000	$1,024,740	$1,024,740
2034	$0	$0	$1,319,000	$1,005,033	$1,005,033
2035	$0	$0	$1,319,000	$985,706	$985,706
2036	$0	$0	$1,319,000	$966,750	$966,750
2037	$0	$0	$1,319,000	$948,159	$948,159
2038	$0	$0	$1,319,000	$929,925	$929,925
2039	$0	$0	$1,319,000	$912,042	$912,042
2040	$0	$0	$1,319,000	$894,502	$894,502
	Sub-Total NPV value =	$70,232,885		$21,649,384	
	Total NPV value =	**$91,880,000**			**$91,880,000**

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Disinfection Alternative 1 - Chlorination/Dechlorination

Opinion of Probable Cost

Item	Quantity	Unit	Unit Cost	Installatiion Factor	Subtotal Cost	Comments
Disinfection System						
Chlorine Contact Tank	1,100	m^3	$1,500	1	$1,650,000	Total Volume: 1,100 m^3
Dechlorination Contact Tank	75	m^3	$2,500	1	$188,000	Total Volume: 75 m^3
Chemical System (Chlorination and Dechlorination)	1	LS	$500,000	1	$500,000	
Instrumentation and Control	1	L.S	$75,000	1	$75,000	
Electrical	1	L.S	$150,000	1	$150,000	
Subtotal Capital Cost					**$2,563,000**	
Engineering (15%)					$384,000	15%
Contigency Cost (5%)					$128,000	5%
Estimating Allowance (20%)					$513,000	20%
General Contractor's Overhead & Profit, Mob.,bond (15%)					$384,000	15%
Total Project Capital Cost (Excluding HST)					**$3,970,000**	

Annual Operation & Maintenance Cost (at Phase 1 Rated Capacity of 30 MLD)

Description	Annual Cost	Comments
Energy	$0	$0.15/kWh,
Chemical Consumption	$10,000	for chlorination and dechlorination
Equipment Maintenance/Replacement	$21,750	3% of equipment cost
Labor	$1,800	$50/hr; 0.1 hrs/d
Total Annual O&M Cost	**$34,000**	

Life Cycle Cost

Economic Factors

Interest rate (%)	4.0%
Inflation rate (%)	2.0%

Cost in Year n = Cost in Current Year x (1+inflation Rate)^(Year n - Current Year)
Present Value = Cost /((1+Interest Rate)^(Year n - Current Year))

NPV					
Year	**Capital Cost**	**NPV Capital Cost**	**Operating Cost**	**NPV Operating Cost**	**Capital and Operating NPV**
	(2020$)		(2020$)		
2020	$0				
2021	$3,970,000	$3,893,654	$34,000	$33,346	$3,927,000
2022	$0	$0	$34,000	$32,705	$32,705
2023	$0	$0	$34,000	$32,076	$32,076
2024	$0	$0	$34,000	$31,459	$31,459
2025	$0	$0	$34,000	$30,854	$30,854
2026	$0	$0	$34,000	$30,261	$30,261
2027	$0	$0	$34,000	$29,679	$29,679
2028	$0	$0	$34,000	$29,108	$29,108
2029	$0	$0	$34,000	$28,548	$28,548
2030	$0	$0	$34,000	$27,999	$27,999
2031	$0	$0	$34,000	$27,461	$27,461
2032	$0	$0	$34,000	$26,933	$26,933
2033	$0	$0	$34,000	$26,415	$26,415
2034	$0	$0	$34,000	$25,907	$25,907
2035	$0	$0	$34,000	$25,409	$25,409
2036	$0	$0	$34,000	$24,920	$24,920
2037	$0	$0	$34,000	$24,441	$24,441
2038	$0	$0	$34,000	$23,971	$23,971
2039	$0	$0	$34,000	$23,510	$23,510
2040	$0	$0	$34,000	$23,058	$23,058
	Sub-Total NPV value =	$3,893,654		$558,058	
	Total NPV value =		**$4,450,000**		**$4,450,000**

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Disinfection Alternative 2 - UV System

Opinion of Probable Cost

Item	Quantity	Unit	Unit Cost	Installatiion Factor	Subtotal Cost	Comments
Disinfection System						
Two (2) Channel UV System (UV3000plus)	2	EA	$550,000	1.5	$1,650,000	Based on Vendor quote
UV Building Cost	150	m^2	$3,000	1	$450,000	Building footprint: 15 m x 10 m =150 m2
Miscellaneous	1	L.S	$200,000	1	$200,000	
Instrumentation and Control	1	L.S	$278,000	1	$278,000	
Electrical	1	L.S	$555,000	1	$555,000	
Subtotal Capital Cost					**$3,133,000**	
Engineering (15%)					$470,000	15%
Contigency Cost (5%)					$157,000	5%
Estimating Allowance (20%)					$627,000	20%
General Contractor's Overhead & Profit, Mob.,bond (15%)					$470,000	15%
Total Project Capital Cost (Excluding HST)					**$4,860,000**	

Annual Operation & Maintenance Cost (at Phase 1 Rated Capacity of 30 MLD)

Description	Annual Cost	Comments
Energy	$32,850	$0.15/kWh, Average 25 kW, 24 hrs/d
Chemical Consumption	$0	for disinfection
Equipment Maintenance/Replacement	$34,400	$420/lamp replacement, 82 lamps
Labor	$1,800	$50/hr; 0.1 hrs/d
Total Annual O&M Cost	**$69,000**	

Life Cycle Cost

Economic Factors

Interest rate (%)	4.0%
Inflation rate (%)	2.0%

Cost in Year n = Cost in Current Year x (1+inflation Rate)^(Year n - Current Year)
Present Value = Cost /((1+Interest Rate)^(Year n - Current Year))

NPV					
Year	**Capital Cost**	**NPV Capital Cost**	**Operating Cost**	**NPV Operating Cost**	**Capital and Operating NPV**
	(2020$)		(2020$)		
2020	$0				
2021	$4,860,000	$4,766,538	$69,000	$67,673	$4,834,212
2022	$0	$0	$69,000	$66,372	$66,372
2023	$0	$0	$69,000	$65,095	$65,095
2024	$0	$0	$69,000	$63,843	$63,843
2025	$0	$0	$69,000	$62,616	$62,616
2026	$0	$0	$69,000	$61,412	$61,412
2027	$0	$0	$69,000	$60,231	$60,231
2028	$0	$0	$69,000	$59,072	$59,072
2029	$0	$0	$69,000	$57,936	$57,936
2030	$0	$0	$69,000	$56,822	$56,822
2031	$0	$0	$69,000	$55,729	$55,729
2032	$0	$0	$69,000	$54,658	$54,658
2033	$0	$0	$69,000	$53,607	$53,607
2034	$0	$0	$69,000	$52,576	$52,576
2035	$0	$0	$69,000	$51,565	$51,565
2036	$0	$0	$69,000	$50,573	$50,573
2037	$0	$0	$69,000	$49,600	$49,600
2038	$0	$0	$69,000	$48,647	$48,647
2039	$0	$0	$69,000	$47,711	$47,711
2040	$0	$0	$69,000	$46,794	$46,794
	Sub-Total NPV value =	$4,766,538		$1,132,530	
	Total NPV value =		**$5,900,000**		**$5,899,000**

B

Appendix B

Evaluation Matrix for Secondary Treatment and Disinfection Alternatives

Table B-1 Detailed Evaluation of Secondary Treatment Options for the Proposed SNF WWTP

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 -Conventional Activated Sludge (CAS)		Alternative 2- Biological Aerated Filter (BAF)		Alternative 3- Biological Nutrient Removal (BNR)	
			Rationale	Score	Rationale	Score	Rationale	Score
Environmental (25%)	Potential Impact on Environmentally Sensitive Features	• - Impact to environmentally sensitive features (e.g. Provincially Significant Wetlands (PSW), Environmental Sensitive Areas (ESA), Environmental Consideration Areas (ECA), Areas of Natural and Scientific Interest (ANSI), significant woodlots, creeks and other designated natural areas as per Official Plans (City of Niagara Falls or Niagara Region) and Niagara Peninsula Conservation Authority or Niagara Parks regulated areas - Maximizes natural buffer	• Similar impact for all options • Potential to mitigate impact through maximizing road right-of-way and trenchless construction. • Option minimizes need to cross environmental features. • High potential to buffer odour, air and noise	●	• Similar impact for all options • Potential to mitigate impact through maximizing road right-of-way and trenchless construction. • Option minimizes need to cross environmental features. • High potential to buffer odour, air and noise	●	• Similar impact for all options • Potential to mitigate impact through maximizing road right-of-way and trenchless construction. • Option minimizes need to cross environmental features. • High potential to buffer odour, air and noise	●
	Impact to Species at Risk	• - Impact to Species at Risk and sensitive aquatic habitats (e.g. proximity to vulnerable/threatened/endangered or locally/regionally rare amphibians, wildlife or fish) - Impacts on sensitive terrestrial flora and fauna habitats	• Due to avoidance of natural features, lower potential for impact to Species at Risk.	●	• Due to avoidance of natural features, lower potential for impact to Species at Risk.	●	• Due to avoidance of natural features, lower potential for impact to Species at Risk.	●
	Potential Effects to Water Features/ Resources	– Impact on surface water levels (short or long-term) – Impact to crossing of floodplains and meander belts (e.g. potential flooding and erosion risk) – Impact on water quality, including nearby water sources and surface/groundwater	• Construction of new infrastructure is required at proposed site to provide the required Phase 1 capacity. There is minimal potential for impacts to natural features during this construction within the proposed site boundaries.	◕	• Construction of new infrastructure is required at proposed site to provide the required Phase 1 capacity. There is minimal potential for impacts to natural features during this construction within the proposed site boundaries.	◕	• Construction of new infrastructure is required at proposed site to provide the required Phase 1 capacity. There is minimal potential for impacts to natural features during this construction within the proposed site boundaries.	◕

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 -Conventional Activated Sludge (CAS)		Alternative 2- Biological Aerated Filter (BAF)		Alternative 3- Biological Nutrient Removal (BNR)	
			Rationale	Score	Rationale	Score	Rationale	Score
	Receiving Waterbody	– Impact on effluent criteria and outfall considerations – Impact to health of receiving waterbody – Minimizes chemical components for treatment – Ability to meet regulatory requirements – Impact on recreational uses – Ability to protect existing water uses	• All options will meet effluent quality requirements • Require chemical usage for phosphorus removal	◕	• All options will meet effluent quality requirements • Require chemical usage for phosphorus removal	◕	• All options will meet effluent quality requirements • Reduced chemicals required for phosphorus removal	●
	Impact on System Overflows	– Maximize opportunities to reduce overflows – Ability to alleviate the existing system and strain on Stanley Avenue Wastewater Treatment Plant and related Sewage Pumping Stations	• All options provide similar peak flow capacity to treat wet weather flows and minimize overflows	◕	• All options provide similar peak flow capacity to treat wet weather flows and minimize overflows	◕	• All options provide similar peak flow capacity to treat wet weather flows and minimize overflows	◕
	Physical Environmental Considerations (Geology, Hydrogeology, Soil/Land Contamination)	– Minimizes environmental crossings – Minimizes time required for contamination review/investigation/ remediation – Subsurface soils and rock characteristics, groundwater levels and water table levels – Level of short or long-term anticipated groundwater impacts (e.g. drilling through water table) – Investigation of potential Hydrogen Sulphide (H_2S)	• Requires investigation of subsurface soils and rock characteristics, groundwater levels and water table levels	◕	• Reduced footprint compared to CAS. • Low potential for contaminated soil	●	• Requires investigation of subsurface soils and rock characteristics, groundwater levels and water table levels	◕
	Climate Change	- Impact on long-term planning - Level of adaptability/resilience - Flexibility in operation and treatment needs - Minimize impact from climate or contributing to climate conditions	• CAS technology provides some flexibility to accommodate extreme conditions due to climate change as infrastructure is designed according to guidelines that provide some conservatism to handle fluctuations in conditions. Compatible for future intensification retrofits as they mature	◕	• BAF offers some operational flexibility during flow and load variations, including improved treatment of dilute and cold wastewater. Limited flexibility for future intensification retrofits.	◑	• BNR technology provides some flexibility to accommodate extreme conditions due to climate change as infrastructure is designed according to guidelines that provide some conservatism to handle fluctuations in conditions. Compatible for future intensification retrofits as they mature	◕

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 -Conventional Activated Sludge (CAS)		Alternative 2- Biological Aerated Filter (BAF)		Alternative 3- Biological Nutrient Removal (BNR)	
			Rationale	Score	Rationale	Score	Rationale	Score
	Overall Environmental Risk	- Potential environmental risk during construction and/or operation - Potential for non-mitigatable impact - Potential for lack of success of the overflow strategy	• Low potential for environmental risk due to minimal environmental features on the site	◕	• Low potential for environmental risk due to minimal environmental features on the site	◕	• Low potential for environmental risk due to minimal environmental features on the site	◕
Social/Cultural (25%)	Community Concerns for Residents/Local Businesses/ Traffic	- Reduces public health and safety concerns - Impact on recreational amenities - Existing and future employment and population areas - Impact of Wastewater Treatment Plant aesthetics - Impact on surrounding properties or public spaces - Impact on travel time during construction - Impact of temporary local disruption to road and public transit traffic - Ability to improve local aesthetics - Nature of adverse effects on roadway - Coordination with planned road work improvements	• Conventional treatment technologies provide a high level of wastewater treatment, so this option does not result in any increased risk to the public or downstream users of the Chippawa Creek.	●	• BAF treatment technologies provide a high level of wastewater treatment, so this option does not result in any increased risk to the public or downstream users of the Chippawa Creek.	●	• BNR treatment technologies provide a high level of wastewater treatment, so this option does not result in any increased risk to the public or downstream users of the Chippawa Creek.	●
	Impact on Indigenous Communities	- Impact during construction and operation • - Impact on short and long-term planning	• No anticipated impact to First Nations communities as construction is limited to existing disturbed sites. Effluent quality will meet all regulations and is not expected to impact First Nations communities downstream.	●	• No anticipated impact to First Nations communities as construction is limited to existing disturbed sites. Effluent quality will meet all regulations and is not expected to impact First Nations communities downstream.	●	• No anticipated impact to First Nations communities as construction is limited to existing disturbed sites. Effluent quality will meet all regulations and is not expected to impact First Nations communities downstream.	●
	Impacts on Archaeological/ Cultural Heritage Features	- Impacts on nearby agricultural lands - Likelihood for impact to heritage homes/properties/landscape - Presence of known archaeological resources/sites, potential impacts on them and ability to mitigate - Number of known archeological sites affected	• New infrastructure to be constructed would be limited to the proposed site, which has little to no remaining archaeological potential.	●	• New infrastructure to be constructed would be limited to the proposed site, which has little to no remaining archaeological potential.	●	• New infrastructure to be constructed would be limited to the proposed site, which has little to no remaining archaeological potential.	●

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 -Conventional Activated Sludge (CAS)		Alternative 2- Biological Aerated Filter (BAF)		Alternative 3- Biological Nutrient Removal (BNR)	
			Rationale	Score	Rationale	Score	Rationale	Score
	Air Quality and Odour Impact	- Impact to surrounding land users - Impact on life cycle air quality associated with overall servicing strategy - Impact on odour from operation of Sewage Pumping Stations, forcemain and/or sanitary sewers - Impact of H2S to create odorous environment - Impact of air quality surrounding the site or official regulations - Incorporation of treatment technologies	• Lower potential to impact air quality due to aeration and smaller tanks compared to BNR.	◕	• Lower potential to impact air quality due to aeration and aerobic condition in the BAF tanks.	◕	• Higher potential to impact air quality due to anaerobic and anoxic conditions in the bioreactors.	◐
	Noise, Vibration and Dust Impact	- Impact of factors: noise, vibration, dust (potential impacts - major, moderate, minor) associated with the impact factors	• The proposed SNF WWTP site has potential to buffer odour, air and noise	●	• The proposed SNF WWTP site has potential to buffer odour, air and noise	●	• The proposed SNF WWTP site has potential to buffer odour, air and noise	●
	Compatibility with Current/ Planned Land Uses	- Suitability of land use designation	• The selected site is a large greenfield area to support siting and flexibility of the SNF WWTP	●	• The selected site is a large greenfield area to support siting and flexibility of the SNF WWTP	●	• The selected site is a large greenfield area to support siting and flexibility of the SNF WWTP	●
	Overall Socio/Cultural Risk	- Cultural heritage or archaeological delays - Impact to community during construction and operation (odour, noise, etc.)	• Good road access for construction and operations • There will be some potential impacts during construction due to additional truck traffic and noise, which can be mitigated with proper construction practices and schedules. Impacts will be limited to SNF WWTP area during construction. Additional nuisance impacts are not expected during normal plant operations.	◕	• Good road access for construction and operations • There will be some potential impacts during construction due to additional truck traffic and noise, which can be mitigated with proper construction practices and schedules. Impacts will be limited to SNF WWTP area during construction. Additional nuisance impacts are not expected during normal plant operations.	◕	• Good road access for construction and operations • There will be some potential impacts during construction due to additional truck traffic and noise, which can be mitigated with proper construction practices and schedules. Impacts will be limited to SNF WWTP area during construction. Additional nuisance impacts are not expected during normal plant operations.	◕

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 -Conventional Activated Sludge (CAS)		Alternative 2- Biological Aerated Filter (BAF)		Alternative 3- Biological Nutrient Removal (BNR)	
			Rationale	Score	Rationale	Score	Rationale	Score
Legal/Jurisdictional (10%)	Approvals/ Coordination	– Potential conflicts or conformity with City of Niagara Falls or Niagara Region Official Plan policies, including Secondary Plans, Master Servicing Plans, and Niagara Peninsula Conservation Authority or Niagara Parks regulations – Compliance with federal, provincial and local plans – Effluent criteria and outfall considerations – Minimize need for environmental approvals for removal of environmental features – Impact to aquatic or natural environments – Reduction of noise and odour impacts – Minimizes jurisdictional requirements (maximize infrastructure within existing regional road right of ways and minimize impact to Ontario Power Generation maintenance/operations)	• Performance of CAS is well understood and there is little complexity or effort expected to obtain approval	●	• Performance of CAS is well understood and there is little complexity or effort expected to obtain approval	●	• Performance of CAS is well understood and there is little complexity or effort expected to obtain approval.	●
	Land Use Suitability	– Compatibility with existing future land use designations – Proximity to physical features (i.e. waterbodies/ highways/railways/residential/recreational)	• Large greenfield area to support siting and flexibility • No changes compared to existing conditions	●	• Large greenfield area to support siting and flexibility • No changes compared to existing conditions	●	• Large greenfield area to support siting and flexibility • No changes compared to existing conditions	●
	Land Acquisition	- Land requirement issues and agency concerns that may arise related to project routes, siting and land acquisition - Site compatibility - Degree of complexity relating to: - Availability of land - Current designated land use - Current ownership - Property acquisition and easement requirements	• No additional land acquisition would be required for a conventional future expansion of the SNF WWTP.	●	• No additional land acquisition would be required for future expansion of the SNF WWTP using BAF technology.	●	• No additional land acquisition would be required for a conventional future expansion of the SNF WWTP using BNR technology.	●

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 -Conventional Activated Sludge (CAS)		Alternative 2- Biological Aerated Filter (BAF)		Alternative 3- Biological Nutrient Removal (BNR)	
			Rationale	Score	Rationale	Score	Rationale	Score
	Worker Safety and Operability	– Accessibility for operation and maintenance	• The selected site has good road access for construction and operations	●	• The selected site has good road access for construction and operations	●	• The selected site has good road access for construction and operations	●
	Overall Legal/ Jurisdictional Risk	– Complexity of land acquisition/ownership – Complexity of approvals/coordination	• Lower risk due to avoidance of environmental constraints and demonstrated full-scale applications.	◕	• Lower risk due to avoidance of environmental constraints and demonstrated full-scale applications.	◕	• Lower risk due to avoidance of environmental constraints and demonstrated full-scale applications.	◕
Technical (20%)	Technical Treatment	- Secondary or tertiary treatment requirements - Effluent Discharge requirements (present/future) - Disinfection options - Flow forecasts - Flexibility to incorporate treatment technologies - Potential for H2S gas during construction - Odour and noise remediation requirements	• Proven technology that will reliably meet all effluent requirements	●	• Proven technology that will reliably meet all effluent requirements	●	• Proven technology that will reliably meet all effluent requirements	●

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 -Conventional Activated Sludge (CAS)		Alternative 2- Biological Aerated Filter (BAF)		Alternative 3- Biological Nutrient Removal (BNR)	
			Rationale	Score	Rationale	Score	Rationale	Score
	– Compatibility/ Impacts to Existing and Future Infrastructure	- Flexibility for future expansion, upgrades or connections - Ability to maximize use of existing infrastructure - Coordination opportunities with planned infrastructure improvements - Integration or Impact with existing utilities and other infrastructure and ability to maintain utilities and infrastructure in service - Utility easements within or in close proximity - Potential for infrastructure to impact recent or planned investments - Minimizes watercourse/highway/railway crossings - Construction in areas with limited access - Proximity and/or conflict with existing infrastructure - Accessibility and safety - Ability to maintain existing services during and following construction - Operational flexibility - Ability to meet future servicing needs for new growth and post 2041 projections - Flexibility with future servicing requirements - Maximize service area	• Compatible with maturing intensification technologies (AGS, etc.) and energy efficiency improvements (MABR) and future tertiary treatment	●	• Proprietary technology with reduced potential for intensification and energy efficiency improvements. Compatible with future tertiary treatment.	◐	• Compatible with maturing intensification technologies (AGS, etc.) and energy efficiency improvements (MABR) and future tertiary treatment	●

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 -Conventional Activated Sludge (CAS)		Alternative 2- Biological Aerated Filter (BAF)		Alternative 3- Biological Nutrient Removal (BNR)	
			Rationale	Score	Rationale	Score	Rationale	Score
	– Biosolids Strategy	– - Close proximity to existing Biosolids Plant - Easy truck access to Biosolids - Ability to pump to Biosolids - Ability to minimize infrastructure needs	• Biosolids are compatible with existing end-use and allow use of familiar thickening (i.e., Gravity Belt Thickener) technology to Region staff	◕	• Biosolids are compatible with existing end-use. Dilute backwash solids requires additional consideration for thickening facility design.	◐	• Reduced biosolids production • Biosolids are compatible with existing end-use and allow use of familiar thickening (i.e., Gravity Belt Thickener) technology to Region staff.	●
	– System Security and Level of Service	- Ability to maintain or enhance operational security – Ability to maintain or enhance service standard for the customer	• All Options provide a similar level of service	●	• All Options provide a similar level of service	●	• All Options provide a similar level of service	●
	– Traffic Management	- Anticipated degree of construction truck traffic management issues during construction – Anticipated level of truck traffic during typical operations and maintenance	• Slightly higher traffic during operation for chemical delivery and sludge haulage	◕	• Slightly higher traffic during operation for chemical delivery and sludge haulage	◕	• Reduced traffic during operation associated with chemical delivery and sludge haulage	●
	Operation & Maintenance	– - Minimizes long-term operation and maintenance requirements - Ease of access to operate and maintain - Provision of emergency access - Deterioration (condensation, salt and H_2S)	• Simple to operate and well understood by Region staff • Significant operational flexibility	●	• Simple to operate • Reduced operational flexibility	◕	• More complex system to operate compared to CAS and less familiar to Region staff	◐
	Overall Technical Risk	- Impact on growth (capacity risk) - Overdesigning and stranding capacity (capacity risk) – - Treatment technology risk - Construction risk - Schedule/timing risk	• Non-proprietary technology with equipment available from multiple vendors	●	• Proprietary technology with limited vendors. Requires pre-selection or pre-purchase	◕	• Non-proprietary technology with equipment available from multiple vendors	●

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 -Conventional Activated Sludge (CAS)		Alternative 2- Biological Aerated Filter (BAF)		Alternative 3- Biological Nutrient Removal (BNR)	
			Rationale	Score	Rationale	Score	Rationale	Score
Financial (20%)	Capital Cost	– Total capital (construction) cost for new infrastructure and/or upgrades for overall servicing strategy – Cost of required/needed property acquisition/easements	• Moderate capital cost	◕	• Moderate capital cost	◕	• Higher capital cost	◐
	Lifecycle Cost (Operation, Resourcing, and Maintenance and Servicing)	– Minimizes operation & maintenance costs – Cost of operation and maintaining the infrastructure	• Moderate O&M and Life Cycle Costs	◕	• Moderate O&M and Life Cycle Costs	◕	• Moderate O&M and Life Cycle Costs	◕
	Cash Flow/Phasing of Costs	– Impact to cash management – Futureproof costing impact (i.e. potential for decisions or treatment options to become obsolete and difficult to replace in the future) – Phasing of costs and impact to DCs and rates	• Capacity and expansion phasing can be accommodated • Flexibility to integrate new technologies as they mature for intensification or energy reduction	●	• Capacity and expansion phasing can be accommodated • Limited flexibility to integrate new technologies for intensification or energy reduction	◕	• Capacity and expansion phasing can be accommodated • Flexibility to integrate new technologies as they mature for intensification or energy reduction	●
	Funding Opportunities	– Developmental Charges – Grants (Federal, Provincial)	• Similar funding opportunities for all technologies	●	• Similar funding opportunities for all technologies	●	• Similar funding opportunities for all technologies	●
	Overall Financial Risk	– Financial risk during construction (cost increase/ uncertainty) – Complexity of solution – Scope increase – Potential impact of unforeseen costs (capital/operations, etc.)	• Technology is mature and well understood to mitigate financial risk.	●	• Technology is mature and well understood to mitigate financial risk.	●	• Technology is mature and well understood to mitigate financial risk.	●
Total Score			●		◕		◕	

Table B-2 Detailed Evaluation of Effluent Disinfection Options for the South Niagara Falls WWTP

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 – Chlorination/Dechlorination		Alternative 2- UV Disinfection	
			Rationale	Score	Rationale	Score
Environmental (25%)	Potential Impact on Environmentally Sensitive Features	• - Impact to environmentally sensitive features (e.g. Provincially Significant Wetlands (PSW), Environmental Sensitive Areas (ESA), Environmental Consideration Areas (ECA), Areas of Natural and Scientific Interest (ANSI), significant woodlots, creeks and other designated natural areas as per Official Plans (City of Niagara Falls or Niagara Region) and Niagara Peninsula Conservation Authority or Niagara Parks regulated areas - Maximizes natural buffer	• Similar impact for all options • Potential to mitigate impact through maximizing road right-of-way and trenchless construction. • Option minimizes need to cross environmental features. • High potential to buffer odour, air and noise	●	• Similar impact for all options • Potential to mitigate impact through maximizing road right-of-way and trenchless construction. • Option minimizes need to cross environmental features. • High potential to buffer odour, air and noise	●
	Impact to Species at Risk	• - Impact to Species at Risk and sensitive aquatic habitats (e.g. proximity to vulnerable/threatened/endangered or locally/regionally rare amphibians, wildlife or fish) - Impacts on sensitive terrestrial flora and fauna habitats	• Due to avoidance of natural features, lower potential for impact to Species at Risk.	●	• Due to avoidance of natural features, lower potential for impact to Species at Risk.	●
	Potential Effects to Water Features/ Resources	– Impact on surface water levels (short or long-term) – Impact to crossing of floodplains and meander belts (e.g. potential flooding and erosion risk) – Impact on water quality, including nearby water sources and surface/groundwater	• There is minimal potential for impacts to natural features during this construction within the new Plant site boundaries.	◕	• There is minimal potential for impacts to natural features during this construction within the new Plant site boundaries.	◕
	Receiving Waterbody	– Impact on effluent criteria and outfall considerations – Impact to health of receiving waterbody – Minimizes chemical components for treatment – Ability to meet regulatory requirements – Impact on recreational uses – Ability to protect existing water uses	• All options will meet effluent quality requirements • Require chemical usage for chlorination/dechlorination • Minimal risk of discharge of chlorinated effluent to the receiving water, through eliminating chlorine residual by dechlorination.	◕	• All options will meet effluent quality requirements • No chemical usage for disinfection • Non-toxic effluent • No risk of chemicals reaching the environment	●

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 – Chlorination/Dechlorination		Alternative 2- UV Disinfection	
			Rationale	Score	Rationale	Score
	Impact on System Overflows	– Maximize opportunities to reduce overflows – Ability to alleviate the existing system and strain on Stanley Avenue Wastewater Treatment Plant and related Sewage Pumping Stations	• Ability to minimize system overflow through the superchlorination (i.e. overdosing chlorine) followed by dechlorination approach during peak wet weather flow conditions.	●	• Potential of system overflows during peak weather flows, as the UV units is limited to the design peak flow and cannot handle excess extraneous flows.	◐
	Physical Environmental Considerations (Geology, Hydrogeology, Soil/Land Contamination)	– Minimizes environmental crossings – Minimizes time required for contamination review/investigation/ remediation – Subsurface soils and rock characteristics, groundwater levels and water table levels – Level of short or long-term anticipated groundwater impacts (e.g. drilling through water table) – Investigation of potential Hydrogen Sulphide (H_2S)	• Requires investigation of subsurface soils and rock characteristics, groundwater levels and water table levels	◕	• Reduced footprint compared to chlorination/dichlorination option. • Low potential for contaminated soil	●
	Climate Change	- Impact on long-term planning - Level of adaptability/resilience - Flexibility in operation and treatment needs - Minimize impact from climate or contributing to climate conditions	• Chlorination/dechlorination technology provides some flexibility to accommodate extreme conditions due to climate change, as the superchlorination (i.e. overdosing chlorine) followed by dechlorination approach can be used during peak wet weather flow conditions.	●	• UV units is limited to the design peak flow and cannot handle excess extraneous flows.	◐
	Overall Environmental Risk	- Potential environmental risk during construction and/or operation - Potential for non-mitigatable impact - Potential for lack of success of the overflow strategy	• Low potential for environmental risk due to minimal environmental features on the site • Effluent chorine residual can be eliminated by dechlorinating	◕	• Low potential for environmental risk due to minimal environmental features on the site • No risk of chemicals reaching the environment	◕

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 – Chlorination/Dechlorination		Alternative 2- UV Disinfection	
			Rationale	Score	Rationale	Score
Social/Cultural (25%)	Community Concerns for Residents/Local Businesses/ Traffic	- Reduces public health and safety concerns - Impact on recreational amenities - Existing and future employment and population areas - Impact of Wastewater Treatment Plant aesthetics - Impact on surrounding properties or public spaces - Impact on travel time during construction - Impact of temporary local disruption to road and public transit traffic - Ability to improve local aesthetics - Nature of adverse effects on roadway - Coordination with planned road work improvements	• May cause public concerns due to the potential formation of disinfection by-products (DBPs); and potential discharge of chlorine residual to the receiving waterbody, however, this can be eliminated by dechlorination. • Some traffic impact for chemical delivery	◕	• No public concerns with operation and safety in the use of UV units, as the use of UV light as a disinfectant does not create any DBP formation and no in-stream chemicals are required to achieve primary disinfection.	●
	Impact on Indigenous Communities	- Impact during construction and operation • - Impact on short and long-term planning	• No anticipated impact to First Nations communities as construction is limited to existing disturbed sites. Effluent quality will meet all regulations and is not expected to impact First Nations communities downstream. • Effluent chorine residual can be eliminated by dechlorination.	◕	• No anticipated impact to First Nations communities as construction is limited to existing disturbed sites. Effluent quality will meet all regulations and is not expected to impact First Nations communities downstream.	●
	Impacts on Archaeological/ Cultural Heritage Features	- Impacts on nearby agricultural lands - Likelihood for impact to heritage homes/properties/landscape - Presence of known archaeological resources/sites, potential impacts on them and ability to mitigate - Number of known archeological sites affected	• Disinfection facility to be constructed would be limited to the proposed site, which has little to no remaining archaeological potential.	●	• Disinfection facility to be constructed would be limited to the proposed site, which has little to no remaining archaeological potential.	●

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 – Chlorination/Dechlorination		Alternative 2- UV Disinfection	
			Rationale	Score	Rationale	Score
	Air Quality and Odour Impact	- Impact to surrounding land users - Impact on life cycle air quality associated with overall servicing strategy - Impact on odour from operation of Sewage Pumping Stations, forcemain and/or sanitary sewers - Impact of H2S to create odorous environment - Impact of air quality surrounding the site or official regulations - Incorporation of treatment technologies	• Potential impact on air quality due to greenhouse gas emission from the traffic for chemical delivery.	◕	• No air quality and odour impact.	●
	Noise, Vibration and Dust Impact	- Impact of factors: noise, vibration, dust (potential impacts - major, moderate, minor) associated with the impact factors	• The proposed SNF WWTP site has potential to buffer odour, air and noise	●	• The proposed SNF WWTP site has potential to buffer odour, air and noise	●
	Compatibility with Current/ Planned Land Uses	- Suitability of land use designation	• The selected site is a large greenfield area to support siting and flexibility of the SNF WWTP	●	• The selected site is a large greenfield area to support siting and flexibility of the SNF WWTP	●
	Overall Socio/Cultural Risk	- Cultural heritage or archaeological delays - Impact to community during construction and operation (odour, noise, etc.)	• Good road access for construction and operations • There will be some potential impacts during construction due to additional truck traffic and noise, which can be mitigated with proper construction practices and schedules. Impacts will be limited to SNF WWTP area during construction. • Effluent chorine residual can be eliminated by dechlorination	◕	• Good road access for construction and operations • There will be some potential impacts during construction due to additional truck traffic and noise, which can be mitigated with proper construction practices and schedules. Impacts will be limited to SNF WWTP area during construction. Additional nuisance impacts are not expected during normal plant operations. • No public concerns with operation and safety in the use of UV disinfection units.	◕

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 – Chlorination/Dechlorination		Alternative 2- UV Disinfection	
			Rationale	Score	Rationale	Score
Legal/Jurisdictional (10%)	Approvals/ Coordination	– Potential conflicts or conformity with City of Niagara Falls or Niagara Region Official Plan policies, including Secondary Plans, Master Servicing Plans, and Niagara Peninsula Conservation Authority or Niagara Parks regulations – Compliance with federal, provincial and local plans – Effluent criteria and outfall considerations – Minimize need for environmental approvals for removal of environmental features – Impact to aquatic or natural environments – Reduction of noise and odour impacts – Minimizes jurisdictional requirements (maximize infrastructure within existing regional road right of ways and minimize impact to Ontario Power Generation maintenance/operations)	• Performance of chlorination/dichlorination disinfection is well understood and there is little complexity or effort expected to obtain approval for the chlorination/dichlorination disinfection system.	●	• Performance of UV disinfection is well understood and there is little complexity or effort expected to obtain approval for the chlorination/dichlorination disinfection system.	●
	Land Use Suitability	– Compatibility with existing future land use designations – Proximity to physical features (i.e. waterbodies/ highways/railways/residential/recreational)	• Suitable land use and close proximity to Chippawa Creek for discharge. • Large greenfield area to support siting and flexibility • No changes compared to existing conditions	●	• Suitable land use and close proximity to Chippawa Creek for discharge. • Large greenfield area to support siting and flexibility • No changes compared to existing conditions	●
	Land Acquisition	- Land requirement issues and agency concerns that may arise related to project routes, siting and land acquisition - Site compatibility - Degree of complexity relating to: - Availability of land - Current designated land use - Current ownership - Property acquisition and easement requirements	• No additional land acquisition would be required for future expansion of disinfection system.	●	• No additional land acquisition would be required for future expansion of disinfection system.	●

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 – Chlorination/Dechlorination		Alternative 2- UV Disinfection	
			Rationale	Score	Rationale	Score
	Worker Safety and Operability	– Accessibility for operation and maintenance	• The selected site has good road access for construction and operations	●	The selected site has good road access for construction and operations	●
	Overall Legal/ Jurisdictional Risk	– Complexity of land acquisition/ownership – Complexity of approvals/coordination	• Lower risk due to avoidance of environmental constraints and demonstrated full-scale applications.	◕	• Lower risk due to avoidance of environmental constraints and demonstrated full-scale applications.	◕
Technical (20%)	Technical Treatment	- Secondary or tertiary treatment requirements - Effluent Discharge requirements (present/future) - Disinfection options - Flow forecasts - Flexibility to incorporate treatment technologies - Potential for H2S gas during construction - Odour and noise remediation requirements	Proven technology that will reliably meet all effluent requirements	◕	• Proven technology that will reliably meet all effluent requirements	◕

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 – Chlorination/Dechlorination		Alternative 2- UV Disinfection	
			Rationale	Score	Rationale	Score
	– Compatibility/ Impacts to Existing and Future Infrastructure	- Flexibility for future expansion, upgrades or connections - Ability to maximize use of existing infrastructure - Coordination opportunities with planned infrastructure improvements - Integration or Impact with existing utilities and other infrastructure and ability to maintain utilities and infrastructure in service - Utility easements within or in close proximity - Potential for infrastructure to impact recent or planned investments - Minimizes watercourse/highway/railway crossings - Construction in areas with limited access - Proximity and/or conflict with existing infrastructure - Accessibility and safety - Ability to maintain existing services during and following construction - Operational flexibility - Ability to meet future servicing needs for new growth and post 2041 projections - Flexibility with future servicing requirements - Maximize service area	• Chlorinatin/dechlorination technology has flexibility for future expansion, upgrades or connection.	◕	• Proprietary technology with reduced potential for energy efficiency improvements. If the UV models are discontinued, the whole UV disinfection system would require upgrades	◐

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 – Chlorination/Dechlorination		Alternative 2- UV Disinfection	
			Rationale	Score	Rationale	Score
	– Biosolids Strategy	- Close proximity to existing Biosolids Plant - Easy truck access to Biosolids - Ability to pump to Biosolids - Ability to minimize infrastructure needs	• Biosolids are compatible with existing end-use and allow use of familiar thickening (i.e., Gravity Belt Thickener) technology to Region staff	●	• Biosolids are compatible with existing end-use and allow use of familiar thickening (i.e., Gravity Belt Thickener) technology to Region staff	●
	– System Security and Level of Service	- Ability to maintain or enhance operational security - Ability to maintain or enhance service standard for the customer	• The two options provide a similar level of service	●	• The two options provide a similar level of service	●
	– Traffic Management	- Anticipated degree of construction truck traffic management issues during construction - Anticipated level of truck traffic during typical operations and maintenance	• Some traffics during operation for chemical delivery	◕	• No traffic during UV operation	●
	Operation & Maintenance	- Minimizes long-term operation and maintenance requirements - Ease of access to operate and maintain - Provision of emergency access - Deterioration (condensation, salt and H_2S)	• Simple to operate and well understood by Region staff • Significant operational flexibility	●	• Simple operation, but less familiar to Region staff compared to the chlorination/dechlorination system	◑
	Overall Technical Risk	- Impact on growth (capacity risk) - Overdesigning and stranding capacity (capacity risk) - Treatment technology risk - Construction risk - Schedule/timing risk	• Technology is mature and well understood to mitigate technical risks.	●	• Proprietary technology with limited vendors. Requires pre-selection or pre-purchase. • Reliability has been a problem in the past, but usage has increased rapidly with improved systems	◕

Criteria	Sub-Criteria	Criteria Indicators	Alternative 1 – Chlorination/Dechlorination		Alternative 2- UV Disinfection	
			Rationale	Score	Rationale	Score
Financial (20%)	Capital Cost	- Total capital (construction) cost for new infrastructure and/or upgrades for overall servicing strategy - Cost of required/needed property acquisition/easements	• Lower capital cost	◕	• Slightly higher capital cost	◕
	Lifecycle Cost (Operation, Resourcing, and Maintenance and Servicing)	- Minimizes operation & maintenance costs - Cost of operation and maintaining the infrastructure - Impact to regional resources - Ease of access to maintain - Provision of emergency access - Minimize total lifecycle cost (combination of capital, property acquisition, operation & maintenance, etc.) - Ability to decommission existing Sewage Pumping Stations - Minimize wastewater infrastructure footprint to reduce impact on climate	• Moderate O&M cost and Lower Life Cycle Costs	◕	• Higher O&M cost and higher Life Cycle Costs	◑
	Cash Flow/Phasing of Costs	- Impact to cash management - Futureproof costing impact (i.e. potential for decisions or treatment options to become obsolete and difficult to replace in the future) - Phasing of costs and impact to Development Charges (DCs) and rates	• Capacity and expansion phasing can be accommodated	◕	• Capacity and expansion phasing can be accommodated with the modular design of UV units.	◕
	Funding Opportunities	- Developmental Charges - Grants (Federal, Provincial)	• Similar funding opportunities for all technologies	◕	• Similar funding opportunities for all technologies	◕

Criteria	Sub-Criteria	Criteria Indicators	**Alternative 1 – Chlorination/Dechlorination**		**Alternative 2- UV Disinfection**	
			Rationale	Score	Rationale	Score
	Overall Financial Risk	- Financial risk during construction (cost increase/ uncertainty) - Complexity of solution - Scope increase - Potential impact of unforeseen costs (capital/operations, etc.)	• Technology is mature and well understood to mitigate financial risk.	●	• Technology is mature and well understood to mitigate financial risk.	●
Total Score			●		◕	