APPENDIX

D DESIGN BASIS TECHNICAL MEMO

MANITOBA WATER SERVICES BOARD
REPORT NUMBER: MWSB 23-53

BOISSEVAIN-MORTON WASTEWATER TREATMENT LAGOON DESIGN

DESIGN BASIS

DECEMBER 18, 2024 FINAL

BOISSEVAIN-MORTON WASTEWATER TREATMENT LAGOON DESIGN

DESIGN BASIS

MANITOBA WATER SERVICES BOARD

TECHNICAL MEMO
FINAL

PROJECT NO.: CA0017817.1877
CLIENT REF:MWSB 23-53
DATE: DECEMBER 18, 2024

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December 18, 2024

FINAL

The Manitoba Water Services Board
Unit 1A-2010 Currie Blvd.
Brandon, Manitoba R7B 4E7

Attention: **Nathan Wittmeier, M.T.S., P.Eng.**

Subject: Community of Boissevain Wastewater Treatment Lagoon Design – Technical Memorandum
Client ref.: MWSB 23-53

Dear Sir:

WSP Canada Inc. is pleased to submit the final version of our technical memorandum for the Community of Boissevain Wastewater Treatment Lagoon Design. This final report contains information that validates the design parameters for the treatment lagoon and verifies the wastewater characterization for the 25-year design period.

Should you have any questions or comments, please feel free to contact the undersigned.

Yours sincerely,

[redacted]

Dana Bredin, P.Eng., PMP
Project Manager

BL/mw
Encl.
cc: Roy San Buenaventura, MWSB; Leo Poulin, Municipality of Boissevain-Morton
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[1] Approval of this document is an administrative function indicating readiness for release and does not impart legal liability on to the Approver for any technical content contained herein. Technical accuracy and fit-for-purpose of this content is obtained through the review process. The Approver shall ensure the applicable review process has occurred prior to signing the document.

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1 INTRODUCTION

1.1 BACKGROUND

The Community of Boissevain currently operates a two-cell aerated wastewater treatment lagoon (WWTL) that was constructed approximately 60 years ago southwest of the Community within the southeast quarter of section 23-3-20 WPM. The lagoon consists of an aerated primary cell and a secondary cell, and the treated effluent is continuously discharged into Cherry Creek. The existing primary cell has an operating depth of 3 m and a total volume of 4,644 m^3, whereas the secondary cell has an operating depth of 1.5 m and a total volume of 21,360 m^3. The primary and secondary cells have a hydraulic retention time (HRT) of 5.3 days and 24.2 days, respectively. The lagoon formerly operated under the Clean Environment Commission (CEC) Order No. 1166 dated January 14, 1988; however, a new Environment Act Licence (EAL) No. 3415 was issued on October 26, 2023, for the proposed WWTL which superseded the CEC Order for the operation of the existing lagoon.

The existing lagoon has historically struggled to meet the CEC Order effluent limits, specifically fecal coliforms and is a continuously discharging facility. As a result, Manitoba Environment and Climate Change required a feasibility study to be completed. The study was completed by Stantec Consulting Ltd. (Stantec) and concluded that the lagoon aeration equipment was undersized and identified that the lagoon berms do not meet current liner requirements as they were constructed with sand, gravel, and organics allowing the wastewater to seep into the surrounding lands. A review of the treated effluent test results completed in December 2017, identified the lagoon exceeds the effluent requirements for BOD_5, Fecal and Total Coliforms. Notably, in all samples collected, the Fecal and Total Coliforms exceeded 2,420 MPN/100 mL, whereas the Environmental Order required Fecal and Total Coliform levels of less than 200 MPN/100 mL and 1,500 MPN/100 mL, respectively.

The Municipality owns approximately 5.39 ha of land in SE 23-3-20 WPM. The northern portion of this land is where the existing lagoon is located, occupying a footprint of approximately 2.87 ha. Therefore, there is 2.52 ha of undeveloped land to the south of the secondary cell. However, the Municipality does not own any additional land surrounding the existing lagoon and the decision was made to select a larger site for the construction of a new WWTL.

A preliminary design for a new WWTL was completed at the proposed site located approximately 6 km north of Boissevain, on property owned by the Municipality (SE 11-4-20 WPM) and north of the existing waste disposal site. The proposed new WWTL consists of two partial-mix aerated cells and two submerged attached growth reactor (SAGR) cells, as well as in-line chemical dosing for phosphorus removal. Both the aerated and SAGR cells require a synthetic liner system (60 mil HDPE). Wastewater will be intercepted at the existing lagoon site and conveyed to the proposed site via a new lift station and forcemain approximately 9,300 m in length. The treated effluent will be conveyed back to Cherry Creek, via another new lift station and forcemain approximately 8,200 m in length. An outfall will be constructed in Cherry Creek as part of this project.

Due to the rising construction costs associated with the new site, the Municipality is now considering expanding the existing lagoon site. The proposed upgrades to the WWTL at the existing site include a new raw water lift station, two SAGR cells, relining and deepening of the existing cells, upgrading the aeration equipment, a new blower building, and in-line chemical dosing for phosphorus removal. The treated effluent will be discharged to Cherry Creek. The existing cells will be relined with either a synthetic liner system or with a surface clay liner. A potential nearby clay source has recently been identified by the community, as part of the geotechnical site investigation the clay will be assessed for its suitability for a clay liner.

1.2 BACKGROUND DOCUMENTS

The following documents were reviewed as part of the study:

- Municipality of Boissevain-Morton Proposed Aerated Lagoon with Post Lagoon Nitrification Pre-Design Report.
- Application for an Environmental Act Licence for an Aerated Lagoon with Post Lagoon Nitrification Wastewater Treatment System for the Community of Boissevain.
- Boissevain Wastewater Lagoon Study and EAP Technical Memorandum #1.
- Boissevain Wastewater Lagoon Study.
- Environment Act Licence No. 3415.
- 2019-2023 Truck Haul Records.
- 2017-2023 Lift Station Pump Hour Records.

2 DESIGN CRITERIA

2.1 PRELIMINARY DESIGN

This section summarizes the design criteria which forms the basis of the preliminary design for the Boissevain Lagoon expansion completed in 2021 by Stantec. The design wastewater flows and wastewater strength to the lagoon were based on a design population of 1,600. The preliminary design flows, literature wastewater characteristics, and influent wastewater strength are presented in **Table 2-1**, **Table 2-2**, and **Table 2-3**, respectively.

Table 2-1: Summary of Preliminary Design Flows

Parameter	Source			Total Flow
	Gravity Sewer	Septage	Holding Tank	
Average Day Flow (m^3/d)	880	2.2	20	902
Design Flow (max month/max hauling condition) (m^3/d)	981	4.8	50	1,036
Raw Wastewater Lift Station Average Hourly Flow (L/s)	10.2	-	-	10.2
Raw Wastewater Lift Station Peak Hour Flow (L/s)	45	-	-	45
Treated Effluent Lift Station Peak Flow (L/s)	25.2	-	-	25.2

Table 2-2: Summary of Preliminary Design Wastewater Characteristics

Parameter	Gravity Sewer - Per capita loading (kg/person/day)	Holding Tank – Sewage Strength (mg/L)
BOD_5	0.090	300
TSS	0.095	350
TKN	0.018	50
NH_3-N	0.014	40
TP	0.003	8

Table 2-3: Summary of Preliminary Design Influent Wastewater Strength

Wastewater Source	Design Flow (m^3/d)	BOD_5 (kg/d)	TSS (kg/d)	TKN (kg/d)	Ammonia (kg/d)	TP (kg/d)
Gravity Sewer	981	144	152	28.8	22.4	4.8
Septage	4.8	33.6	72	3.36	0.72	1.2
Holding Tank	50	15	17.5	2.5	2	0.4
TOTAL	1,036	193	242	35	25	6.4

2.2 REGULATORY REVIEW

The proposed lagoon facility will operate under EAL No. 3415, dated October 26, 2023. The lagoon effluent will be required to meet the limits as presented in **Table 2-4** and **Table 2-5** and in fact these limits already apply to the existing lagoon. The Environmental Compliance and Enforcement Branch (ECEB) has been in communication with the Municipality about the how these limits apply to the existing lagoon and have requested an operational plan on how the Municipality is going to meet the requirements. At the request of the Municipality, WSP has been in communication with the ECEB to discuss the matter. WSP intends to submit an operational plan that focuses on sampling along the discharge route to determine if the limits not being met at the end-of-pipe are met as the partially treated effluent traverses along the route.

Table 2-4: Effluent Limits

Parameter	Limit
$CBOD_5$	25 mg/L
Fecal Coliform	200 CFU per 100 mL
TSS	25 mg/L
Total Phosphorus	1.0 mg/L

Table 2-5: Effluent Ammonia Limits

Effluent pH	Effluent Total, Ammonia expressed as N (mg/L)	Effluent pH	Effluent Total, Ammonia expressed as N (mg/L)	Effluent pH	Effluent Total, Ammonia expressed as N (mg/L)
6.5	48.83	7.4	22.97	8.3	4.71
6.6	46.84	7.5	19.89	8.4	3.88
6.7	44.57	7.6	17.03	8.5	3.2
6.8	42	7.7	14.44	8.6	2.65
6.9	39.16	7.8	12.14	8.7	2.2
7	36.09	7.9	10.13	8.8	1.84
7.1	32.86	8	8.41	8.9	1.56
7.2	29.54	8.1	6.95	9	1.32
7.3	26.21	8.2	5.73		

2.3 POPULATION PROJECTION

Based on the available data, the Community of Boissevain population has fluctuated over the past 17 years. Since the 2021 Census was released, the Community has opened a 35 unit assisted living facility and has developed 10 additional residential lots. Recently, the Municipality has been working with a group that is looking to build additional housing and attract industry. As a result, a design population of 2,000 has been selected by the Municipality, which results in an annual growth rate of 0.8% over the design period. The historical and proposed population projection is summarised in **Table 2-6**.

Table 2-6: Population Projection

Year	Population	Percent Change (%)	Annual Growth Rate (%)
2006	1,497	-	-
2011	1,572	5.0	1.0
2016	1,656	5.3	1.1
2021	1,567	-5.4	-1.1
2022[1]	1,611	2.8	2.8
2023[1]	1,656	2.8	2.8
2048 (projected)	2,000	20.7	0.8

[1] **Population data provided by the Municipality**

2.4 WASTEWATER GENERATION

2.4.1 LIFT STATION DRAWDOWN TEST

The main lift station is located adjacent to the existing lagoon on the west side. The main lift station receives all of the Community of Boissevain's wastewater and conveys it to the lagoon. The lift station was partially refurbished in October 2023 and currently consists of two Flygt NP 3102 MT submersible pumps, with 3.7 kW (5 hp), 220V motors. The lift station building also includes an overflow manhole outfitted with a standby pump which is utilised when inflow exceeds the capacity of the lift station.

The Public Works staff for the Municipality records daily pump hour readings. The Municipality provided WSP with data over the years of 2018-2023. WSP completed a drawdown test at the lift station on December 19, 2023, to calculate an annual wastewater volume generated by the Community. The results of the drawdown testing are presented in **Appendix A.** From the drawdown test, the pumping rates of pumps 1 and 2 were calculated to be 16.39 L/s and 18.54 L/s, respectively. Two cycles were also recorded with both pumps operating simultaneously, the resulting pumping rate was 28.50 L/s. A conservative pumping rate of 20 L/s, was selected for both pumps. **Table 2-7** documents the annual pump hours for each pump and the resulting pumping volumes for the six-year period from 2018-2023.

Table 2-7: Lift Station Annual Pump Hours and Wastewater Volumes

Year	Pump 1 Annual Hours	Pump 2 Annual Hours	Total Volume Pumped (m^3)
2018	898.2	1132.5	146,210
2019	1483.2	1809.0	237,038
2020	1139.6	1503.7	190,317
2021	966.0	1146.5	152,100
2022	1701.3	2759.0	321,141
2023	1691.5	831.8	191,620

Public Works staff noted during wet periods both pumps often run simultaneously for 24 hours or more. During the wet periods a third (standby) pump is installed for use. The pump draws from an overflow manhole adjacent to the lift station barrel. Prior to 2022, the community used two Honda WT30X 75 mm discharge trash pumps to assist during wet periods. WSP determined each of the trash pumps were capable of discharging 15 L/s. In 2022, the Community replaced the trash pumps and installed a self-priming FTT 20-40 pump. The Community has not had to operate the new standby pump since being installed, however, it was determined that the theoretical discharge rate of this pump is 35 L/s under the current operational parameters.

2.4.2 WASTEWATER GENERATION

The annual wastewater generation rates were calculated using the total volume pumped (by the lift station) and the corresponding population. The population was linearly interpolated between census years. The annual wastewater generation rate was compared to the annual metered water consumption rates of the Community. These values are presented in **Table 2-8**.

The Stantec study selected a design wastewater generation rate of 550 L/c/d, which breaks down to 315 L/c/d from domestic wastewater and 235 L/c/d from infiltration and extraneous inflows. At the time the Stantec study was completed lift station pump hours were only available for November 2017.

Table 2-8: Comparison of Boissevain Wastewater Generation Rates and Annual Water Consumption

Year	Population	Annual WW Generation Rate (L/c/d)	Annual Water Consumption (L/c/d)
2018	1,620	242	263
2019	1,603	392	267
2020	1,586	314	283
2021	1,567	252	277
2022	1,621	531	249
2023	1,675	317	222
	AVERAGE	341	261
	MAX	531	283

From a review of the annual average wastewater generation rates from 2018-2023, these rates vary from 242 L/c/d to 531 L/c/d. For the purpose of determining a design wastewater generation rate, 531 L/c/d will be selected, which equates to 856 m^3/d using the average population.

The data from 2018-2023 can also be presented in an average monthly flow over this time period, as shown in **Figure 2-1**. When organized and illustrated in this manner, it becomes clear that there are periods when there is a base monthly flow (without influence from inflow and infiltration) and there are periods with substantial inflow and infiltration experienced in the collection system. From **Figure 2-1**, a base flow of 421 m^3/d was selected, as indicated by the blue line. When 421 m^3/d is converted to a per capita flow, it is identical to the average annual water consumption of 261 L/c/d, which provides a high level of confidence that it is an accurate base flow.

During wet years such as 2022, the Community experienced a maximum day flow of 1907 m^3/d, which when the base flow is subtracted results in a maximum inflow and infiltration rate of 1486 m^3/d. Comparing the base flow to the average month flow, there are three distinct periods in which the average month flow exceeds the base flow. The periods run from May 2019 – November 2020, March 2022 – December 2022, and April 2023 – December 2023 Therefore, the selected design wastewater generation rate of 531 L/c/d can be further broken down into a domestic wastewater generation rate component of 261 L/c/d and inflow and infiltration rate component of 270 L/c/d.

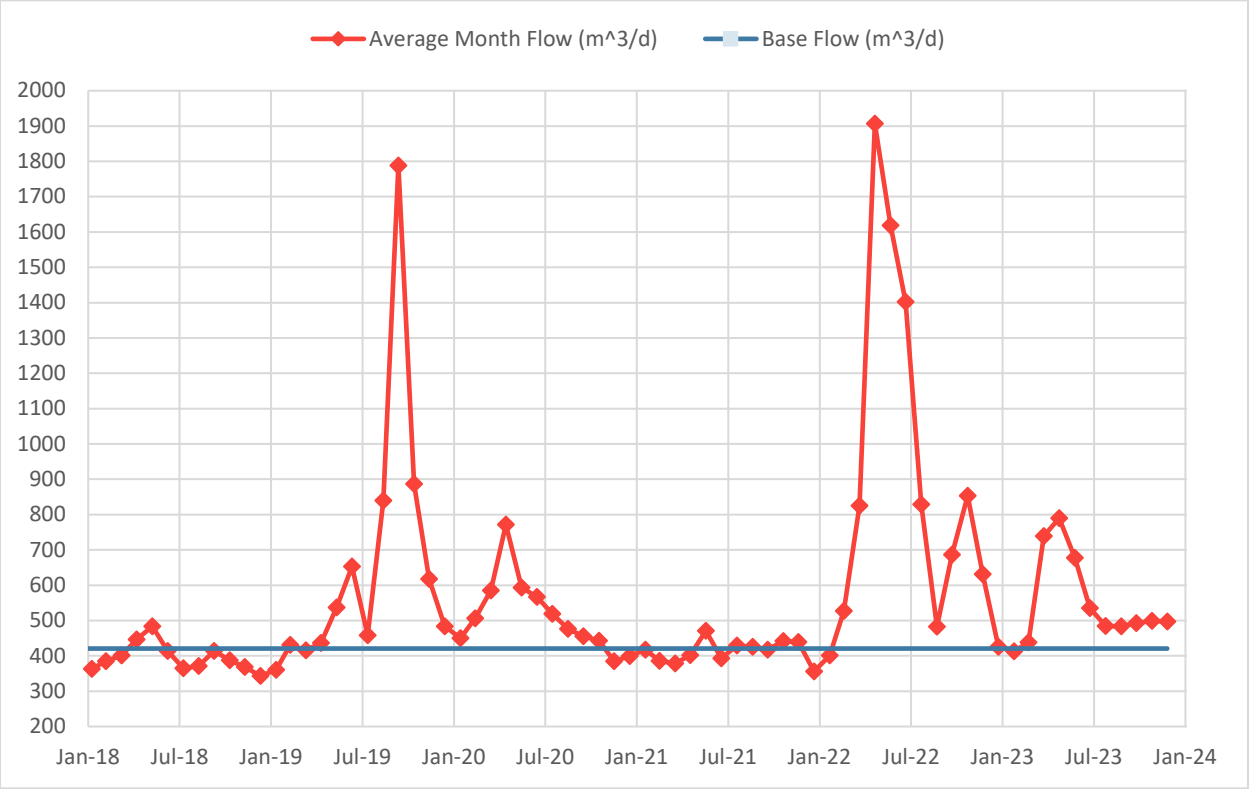

Figure 2-1: Average Monthly Wastewater Flow from 2018-2023 for the Community of Boissevain

2.4.3 PROJECTED LAGOON HYDRAULIC LOADING

All wastewater collected by the Community's sewer system flows by gravity to the lift station. The following criteria were selected to determine the projected hydraulic loading to the lagoon:

- Wastewater generation rate of 531 L/c/d (261 L/c/d - wastewater generation, 270 L/c/d – inflow and infiltration)
- Design population of 2,000

The average day flow to the lagoon was calculated as follows:

$$Average\ daily\ flow: 531\ L/c/d\ *\ 2000\ people * 1/1000\ m^3/L = 1{,}062\ m^3/d\ (12\ L/s)$$

Therefore, the design average daily flow to the lagoon is 1,062 m^3/d. The average daily design flow breaks down to

The theoretical max month flow to the lagoon was calculated as follows:

$$Max\ month\ day\ flow: (1.71 * 261\ L\ /\ c\ /\ d + (270\ L/c)/d) *\ 2000\ people * 1/1000\ m^3/L$$
$$= 1{,}432\ m^3/d\ (17\ L/s)$$

2.4.4 PROJECTED RAW WASTEWATER LIFT STATION

The raw wastewater lift station will be designed based on the wastewater generation rate. Peaking factors were applied to the average day flow to determine the max month, max day, and peak hour flows. As calculated in Section 2.4.3, the average day flow is 11 L/s, this results in a wastewater generation rate of 5.4 L/s and inflow and infiltration rate of 5.6 L/s. The max day month and max day flows were calculated as follows:

$$Max\ Month\ Flow: MDPF * AADF + Inflow\ =\ 1.71 * 6.0\ L/s + 6.3\ L/s = 17\ L/s$$

$$Max\ Day\ Flow: MMPF * AADF = 3.03 * 6.0\ L/s + 6.3\ L/s + 30\ L/s = 55\ L/s$$

Harmon's Peaking Factor was applied to the average day flow plus the Honda standby pumps pumping rate to determine the peak hour flow. Harmon's Peaking Factor and the peak hour flow were calculated as follows:

$$Harmon's\ Peaking\ Factor: 1 + \left(\frac{14}{4 + \left(\left(\frac{2000}{1000} \right)^{.5} \right)} \right) = 3.59$$

$$Peak\ Hour\ \ Flow: HPF * AADF + Inflow + Standby\ Pump = \ 3.59 * 6.0\ L/s + 6.3\ L/s + \ 30\ L/s = 58\ L/s$$

The design flows for the raw wastewater lift station are presented in **Table 2-9.**

Table 2-9: Raw Wastewater Lift Station Design Flows

Parameter	Theoretical Value
Max Month Peaking Factor (MMPF)	1.71
Max Day Peaking Factor (MDPF)	3.03
Harmon's Peaking Factor (HPF)	3.59
Standby Pump (2*15 L/s)	30 L/s
Average Annual Day Flow (AADF)	12 L/s
Max Month Flow (MMF)	17 L/s
Max Day Flow (MDF)	55 L/s
Peak Hour Flow (PHF)	58 L/s

2.4.5 PROJECTED TREATED EFFLUENT LIFT STATION

The lagoon acts as reservoir that buffers the incoming flows. As a result, the peak hour treated effluent flows will be less than the peak hour raw wastewater flow rate. The peak hour treated effluent flow rate was calculated as follows:

$$Peak\ Hour\ \ Flow:\ AADF + Standby\ pump = \ 12\ L/s + 30\ L/s = 42\ L/s$$

The design flows for the effluent lift station are presented in **Table 2-10**.

Table 2-10: Effluent Lift Station Design Flows

Parameter	Theoretical Value
Standby pump (2*15 L/s)	30 L/s
Average Annual Day Flow (AADF)	12 L/s
Peak Hour Flow (PHF)	42 L/s

2.4.6 COMPARISION OF THEORETICAL AND OBSERVED FLOWS

The lift station pumping hours were analysed to identify the max average day, max month, and max day pumping conditions. The observed conditions were compared to the calculated theoretical flow conditions, for the existing conditions. The calculations presented in the previous section were adjusted to account for the 2023 population of 1,656. The theoretical and observed values are presented in **Table 2-11.**

Table 2-11: Comparison of Theoretical and Observed Flow Conditions – Existing Conditions

Parameter	Theoretical Flow	Observed Flow	Date Observed
Max Average Day Flow	879 m^3/d (10 L/s)	881 m^3/d (10 L/s)	2022
Max Month Flow	1,432 (17 L/s)	1,900 m^3/d (22 L/s)	May 2022
Max Day Flow	4,714 m^3/d (55 L/s)	5,054 m^3/d (59 L/s)	May 22, 2022
Peak Hour Flow	58 L/s	N/A	N/A

As shown in the table above, the theoretical and observed flow values are consistent for the maximum average day flow. However, the observed conditions exceed the theoretical values for all other situations. The RM records pump hours on a daily basis, as a result a peak hour flow could not be identified. It is possible that the peak hour flow exceeds the observed max day flow. However, when the max day flow was observed, the pumps ran continuously for 24 hours with both trash pumps operating. As such, the max day flow accounts for the peak hour flow. Therefore, it was determined to select the observed flow conditions as the design criteria.

To determine the projected design flow conditions (based on the observed flow), a population growth rate of 20.8% was applied to the observed flow data. The data was then compared to the previously calculated theoretical flow for the design population of 2,000. The comparison of theoretical and observed flow conditions for the projected population are presented in **Table 2-12**.

Table 2-12: Comparison of Theoretical and Observed Flow Conditions - Projected

Parameter	Theoretical Flow	Observed Flow
Max Average Day Flow	1,062 m^3/d (12 L/s)	1,064 m^3/d (12 L/s)
Max Month Flow	1,294 (15 L/s)	2,295 m^3/d (27 L/s)
Max Day Flow	4,505 m^3/d (52 L/s)	6,104 m^3/d (70 L/s)
Peak Hour Flow	52.1 L/s	N/A

To be conservative, the projected observed flow values will be used for the design basis for the lagoon and lift station design.

2.5 PROJECTED ORGANIC LOADING

2.5.1 GRAVITY SEWER

The per capita loadings were applied to the design population of 2,000 people, to determine the design loading from the gravity sewer. The per capita loadings are based on literature values for piped domestic sewage by Metcalf and Eddy **(Table 3-16, 2014)**. These design organic loading and per capita loading are presented in **Table 2-13**

Table 2-13: Gravity Sewer Organic Loading

Parameter	Per capita loading (kg/person/day)	Design Loading (kg/d)
BOD_5	0.077	154.0
TSS	0.074	148.0
TKN	0.013	26.0
NH_3-N	0.0077	15.4
TP	0.0021	4.2

2.5.2 TRUCK HAUL

WSP was provided with truck haul records for the Boissevain Lagoon from 2018 to 2023, and the data totals are presented in **Table 2-14**. Upon review of the data, it was observed that the volume of truck haul to the lagoon decreased by approximately 40% between 2019 and 2021. The decrease in truck haul is due to the opening of a new lagoon at Bower Lake, which accepts the wastewater from the surrounding cottage communities, that was previously hauled to the Boissevain Lagoon. Southwest Vac is the sole septic truck company in the area that hauls to the Boissevain Lagoon. The Community and Southwest Vac confirmed all wastewater hauled to the lagoon is from holding tanks.

Table 2-14: Truck Haul Annual Volumes

Year	Volume (m^3)
2018	1,715
2019	1,874
2020[1]	572
2021	1,069
2022	1,168
2023	1,011
[1] Records provided from January to June	

The truck haul data was further analysed to determine the peak day loading to the lagoon, as shown in **Table 2-15**. When determining the peak daily loading to the lagoon, the years 2018 and 2019 were omitted because they represent an outdated hauling situation. During the review of truck haul data, it was observed that sewage is hauled to the lagoon a few times a month, as a result, a peak day truck haul volume was selected rather than the average peak month volume. The resulting peak day truck haul volume is 57 m^3/d, which equates to 10 loads in one day. To account for growth in the Municipality an annual growth rate of 0.3% was applied to the truck haul, therefore, a design truck haul volume of 62 m^3/d was selected.

Table 2-15: Peak Day Truck Haul

Year	Date	Volume (m^3)
2018	October 9	97
2019	October 8	124
2020[1]	May 5	48
2021	December 14	48
2022	August 23	57
2023	September 19	44
[1] Records provided from January to June		

The holding tank wastewater was characterized as high strength domestic wastewater. The concentrations are based on literature values for high strength untreated domestic wastewater by Metcalf and Eddy (Table 3-18, 2014). The design organic loading and concentrations for holding tank sewage are presented in **Table 2-16**.

Table 2-16: Holding Tank Organic Loading

Parameter	Concentration (mg/L)	Design Loading (kg/d)
BOD_5	400	24.8
TSS	389	24.1
TKN	69	4.3
NH_3-N	41	2.5
TP	11	0.68

2.6 DESIGN CRITERIA SUMMARY

Table 2-17 summarizes the design parameters used to estimate the projected flows and wastewater loading to the lagoon for the 25-year design horizon. The design parameters include the projected population, per capita flows, organic loadings, and peaking factors. The per capita flows were based on historical flow data and population. The max hour peaking factor was calculated based on Harmon's equation. The organic loadings were based on typical literature values for domestic wastewater for the gravity sewer and high strength wastewater for the holding tanks**.** **Table 2-18** and **Table 2-19** provide a summary of the projected flows and organic loading to the lagoon.

Table 2-17: Design Parameters

Parameter		Design Value
Population		2,000
Flows		
Wastewater Generation Rate, L/c/d		261
Inflow and Infiltration Rate, L/c/d		270
Monthly Peak Factor		1.71
Max Day Peak Factor		3.03
Max Hour Peak Factor		3.59
Organic Loading		
BOD	Gravity Sewer, kg/c/d	0.077
	Holding Tank, mg/L	400
TSS	Gravity Sewer, kg/c/d	0.074
	Holding Tank, mg/L	389
TKN	Gravity Sewer, kg/c/d	0.013
	Holding Tank, mg/L	69
NH_3-N	Gravity Sewer, kg/c/d	0.0077
	Holding Tank, mg/L	41
TP	Gravity Sewer, kg/c/d	0.0021
	Holding Tank, mg/L	11

Table 2-18: Hydraulic Loading Summary

Parameter	Flow
Lagoon	
Average Day Flow	1,064 m^3/d
Max Month Flow	2,295 m^3/d
Raw Water Lift Station	
Average Day Flow	12 L/s
Max Month Flow	27 L/s
Max Day Flow	70 L/s
Peak Hour Flow	70 L/s
Treated Effluent Lift Station	
Average Day Flow	12 L/s
Peak Hour Flow	42 L/s

Table 2-19: Organic Loading Summary

Source	BOD_5 (kg/d)	TSS (kg/d)	TKN (kg/d)	NH_3-N (kg/d)	TP (kg/d)
Gravity Sewer	154.0	148.0	26.0	15.4	4.2
Holding Tank	24.8	24.1	4.3	2.5	0.7
Total	178.8	172.1	30.3	17.9	4.9

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- Statistics Canada, “Boissevain Population Centre Census Profile”, Ottawa, 2023.
- Stantec Consulting Ltd., “Application for an Environmental Act Licence for an Aerated Lagoon with Post Lagoon Nitrification Wastewater Treatment System for the Community of Boissevain”, Winnipeg, 2021.

APPENDIX

A DRAWDOWN TEST RESULTS

Pump Draw Down Calculator

Project:	Boissevain Lagoon Detailed Design
Project No:	CA0017817.1877
Date:	23-12-19
Completed by:	BL/CM

Input Data

Wet well diameter [m]	2.6	m
Wet well dimensions [m]	n/a	m, length
	n/a	m, width
Wet well area [sq.m.]	5.309	sq.m.
Number of pumps	2	
Measured outer diameter		m, W to E
Measured outer diameter		m, N to S

Pipes in pumping volume

150 dia	A=		sq.m.	Pump 1 FM
150 dia	A=		sq.m.	Pump 2 FM
150 dia	A=		sq.m.	Incoming FM
	Atotal=	0.0000	sq.m.	

Lafarge MH size [mm]	Actual Inner Diameter [mm]	Wall thickness [mm]	Outer Diameter [mm]
1500	1524	152	1828
1800	1829	178	2185
2100	2134	203	2540
2400	2438	229	2896
3000	3048	279	3606

A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R
Pump	Start time	Start depth [ft]	Start depth [m]	End time	Ending depth [ft]	Ending depth [m]	Run Time [s]	Fill Time [s]	Uncorrected Pumped Volume [cu.m.]	Uncorrected Pumping Rate [L/s]	Inflow Rate [L/s]	Corrected Pumping Rate [L/s]	Corrected Pumped Volume [L]	Totalizer [cu.m.]	Totalizer Pumped Volume [cu.m.]	Difference in Pumped Volume (P-N) [cu.m.]	Percent Difference in Pumped Volume (P/N-1)
2	9:46:01	4.51	1.37	9:50:26	2.49	0.76	265		3.27	12.33		18.60	4,928.17		-	-	-
								522			6.26						
1	9:59:08	4.51	1.37	10:05:06	2.48	0.76	358		3.28	9.18		15.71	5,625.74		-	-5625.74	-100.0%
								482			6.82						
2	10:13:08	4.51	1.37	10:17:43	2.49	0.76	275		3.27	11.89		18.70	5,142.93		-	-5142.93	-100.0%
								-37063			0.11						
1			0.00			0.00	00		0.00	#DIV/0!		#DIV/0!	#DIV/0!		-	#DIV/0!	#DIV/0!
								38498			0.19						
2	10:41:38	4.51	1.37	10:46:11	2.49	0.76	273		3.27	11.97		18.50	5,049.92		-	-5049.921002	-100.0%
								501			6.52						
1	10:54:32	4.51	1.37	10:59:57	2.49	0.76	325		3.27	10.06		16.55	5,378.72		-	-5378.72	-100.0%
								506			6.46						
2	11:08:23	4.51	1.37	11:12:51	2.49	0.76	268		3.27	12.20		18.38	4,925.02		-	-4925.02	-100.0%
								554			5.90						
1	11:22:05	4.51	1.37	11:27:16	2.48	0.76	311		3.28	10.56		16.89	5,254.31		-	-5254.31	-100.0%
								488			6.76						
1+2	11:35:24	4.52	1.38	11:37:58	2.43	0.74	154		3.38	21.96		28.43	4,377.56		-	-4377.56	-100.0%
								546			6.16						
1+2	11:47:04	4.51	1.37	11:49:32	2.46	0.75	148		3.32	22.41		28.58	4,229.75		-	-4229.75	-100.0%
								538			6.17						
2	11:58:30	4.51	1.37				-43110		7.30	-0.17		#DIV/0!	#DIV/0!		-	#DIV/0!	#DIV/0!
								00			#DIV/0!						
1							00		0.00	#DIV/0!		#DIV/0!	#DIV/0!	-	-	-	-
																Average	#DIV/0!

	Measured		Totalizer		Commissioning	
Pump 2 Average [L/s]	18.54	L/s	#DIV/0!	L/s		L/s
Pump 1 Average [L/s]	16.39	L/s	#DIV/0!	L/s		L/s
Pump 1 + 2 Average [L/s]	28.50	L/s				

Comparison of actual recorded pump run time and the hour meters

Pump	Actual		Actual	Hour Meters	Meters/Actual
1	1296 s	=	0.360 h	0.400	111.1%
2	1383 s	=	0.384 h	0.400	104.1%

*time does not include missing run time for

Hour meters

	P1	P2	
Start time	31830.8	31467.2	enter in the hour meters start and end for the test
End time	31831.2	31467.6	enter in the hour meters start and end for the test
	0.4	0.400	
	1440	1440.0	

Milltronics depth check

Milltronics [m]	Measurement [m]	Difference [m]
1.37	6.24	
0.76	6.84	
0.61	0.60	0.01

confirmation that the Milltronics is operating correctly; use a tape measure to record the liquid level at a specific measurement given by the Milltronics (usually pump start)
confirmation that the Milltronics is operating correctly; use a tape measure to record the liquid level at a specific measurement given by the Milltronics (usually pump stop)

Use a tape measure to record the top to bottom measurement of the lift station and note on this spreadsheet as well