Jacobs

Tasks 1 & 2: Wastewater Treatment Plant and Collection System Capacity Evaluation

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Project name:	Northport/Leelanau Township Utilities Authority (NLTUA) Capacity Evaluation
Project no:	378017A1
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Executive Summary

Jacobs and the Northport/Leelanau Township Utilities Authority (NLTUA) have worked together for over a decade to create a mutually beneficial partnership. Jacobs was retained for the NLTUA Capacity Study to evaluate the existing capacity of the wastewater treatment plant (WWTP) and sanitary sewer collection system. Task 1 of the project entailed Jacobs reviewing existing data to identify gaps for additional data requests or collection from NLTUA. Task 2 involved Jacobs evaluating the existing capacity of both the WWTP and the collection system. This technical memorandum summarizes key workflows and takeaways from the evaluation.

Wastewater Treatment Plant

Jacobs evaluated historical data to determine current conditions and performance of the WWTP, followed by an evaluation of the capacity of each WWTP unit process. A SUMO process model of the WWTP was developed to confirm the biological treatment capacity and quantify how much additional capacity remains underutilized in accordance with the design requirements of the regulatory authority Michigan Department of Environment, Great Lakes and Energy (EGLE). The evaluation had a focus on unit process treatment capacity and redundancy to allow for future planning should upgrades be required to accommodate future growth.

Major findings of the WWTP evaluation include:

- The plant cannot support an increase in current peak flows due to hydraulic constraints of the polishing filters and the 3" effluent flow meter. Peak flows occur infrequently but are still an important design consideration. The peak flows Jacobs observed in the historical data do not directly violate the WWTP's permit, however they can result in operational challenges and damage such as filter media washout and clogged pipes, which could indirectly cause a violation of the permitted effluent limits.
- The biological treatment process, including the pre-anoxic zone and moving bed biofilm reactor (MBBR), can support up to a 2.5-fold increase in maximum month influent pollutant loads. However, secondary treatment solids separation processes, including cavitation air flotation (CAF) and the secondary settling zone, are likely already overloaded and improvements are needed for the overall secondary treatment process to meet its potential.
- A new effluent limit for total inorganic nitrogen (TIN) as low as 5.0 mg/L may be added to the WWTP's discharge permit at some point in the future. New or more stringent limits could reduce the influent

pollutant loads the WWTP can treat while maintaining compliance, thereby impacting the capacity of the WWTP. Historical data indicate a 5.0 mg/L TIN limit would not be met under current operating conditions and evaluating the operational and process configuration changes required to meet such a limit is beyond the scope of this evaluation.

To accommodate future treatment of additional flows and loads, filtration capacity should be expanded to meet peak hour conditions associated with NLTUA's long-term capacity goals. As a short-term measure, automated hydraulic equalization using Treatment Basin #2 could be implemented to mitigate peak flow impacts and reduce the risk of permit noncompliance. Additional recommended improvements include upgrades to the secondary solids separation processes, enhancements to screening reliability, repairs and operational adjustments within the MBBR system, and improvement to the filter reject piping and effluent flow metering.

Collection System

Jacobs prepared a hydraulic model of Northport's collection system in PCSWMM. Provided drawings and system data were used to model summer and winter flow conditions to determine hydraulic capacity and system limitations. Six scenarios were run in PCSWMM to model three weather events in the summer (busy-season) and winter (off-season). Model results show that dry weather conditions do not pose any capacity issues or cause flooding in the system. Results for the 2-year and 5-year storm revealed projected flooding and capacity issues are concentrated along North Mill Street, North and South Shore Drive, North Bay Street, and South Shabwsasung Street. Manholes along these streets are projected to have surface flooding and houses along these streets as well as a handful of other intersections in the Village are at risk of basement backups. The summer is projected to have a slightly larger flood volume (~0.7% more) than winter, but other metrics show minor seasonal variance.

Part 1: Wastewater Treatment Plant

1. Introduction

1.1 Background

The Northport WWTP was established in 2008 and has a design flow of 0.132 million gallons per day (mgd). The flows into the plant are seasonal, with high population flux between summer and winter months due to summer tourism. The WWTP serves the area within the Village of Northport and Leelanau Township and is owned by the Northport/Leelanau Township Utilities Authority (NLTUA or the Authority). The plant is permitted for a maximum daily effluent discharge flow of 405,000 gallons per day (gpd). Refer to Appendix A for the current groundwater discharge permit. Jacobs has been operating and maintaining the WWTP and collection system since 2008 and in 2018 the contract was renewed for an additional 10 years.

1.2 Scope of Work

The Authority retained Jacobs to undertake a capacity evaluation of the Northport WWTP. The scope of Tasks 1 and 2 includes:

- Analyze historical data and characterize current influent flows and loads
- Verify the treatment capacity of the WWTP using process modeling and estimate influent flows and loads that can be accommodated with the existing treatment infrastructure
- Quantify how much additional capacity remains to serve potential additional development within the service area

1.3 Project Location and Plant Overview

Figure 1-1 shows the project location and Figure 1-2 shows an overview of the WWTP.

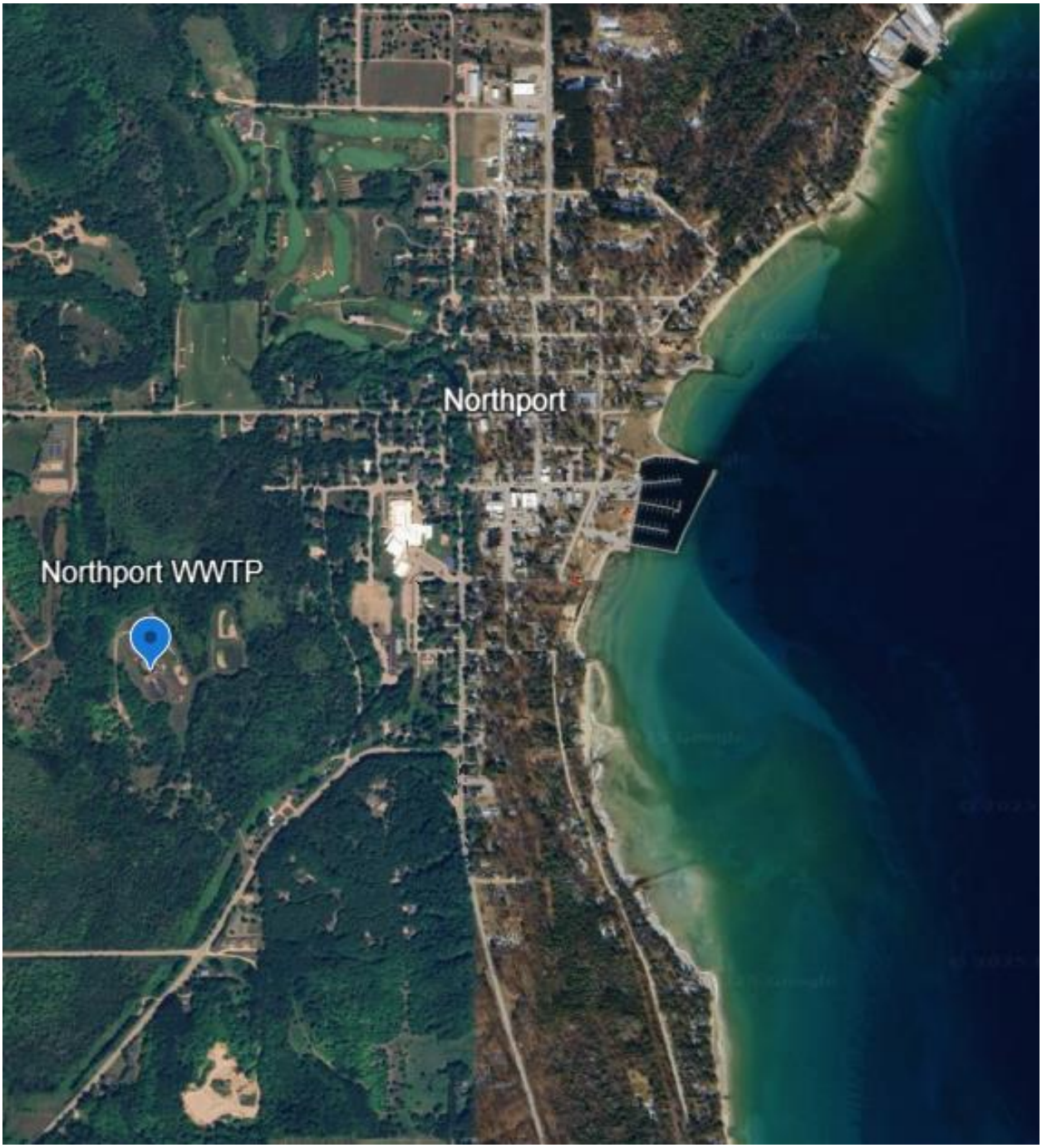

Figure 1-1. Project Location

Figure 1-2. WWTP Overview

1.4 Capacity Evaluation Approach

The following summarizes the approach taken to complete the capacity evaluation of the Northport WWTP:

- Historical wastewater data was analyzed to determine historical wastewater flows and loads for both the summer (busy-season) and the winter (off-season).
- A special sampling plan was carried out to better characterize the WWTP influent and improve model calibration.

- A SUMO process model of the WWTP was developed and calibrated based on the plant data.
- The plant process model was used to determine the existing treatment capacity of the biological treatment process and quantify additional capacity available.
- The capacity of each unit process was evaluated.
- Based on the findings of the capacity evaluation, recommendations are made to allow for future planning and accommodate future growth.

SUMO is a commercially available whole-plant process modeling software with both steady-state and dynamic simulation capabilities. Modules for individual processes within a SUMO model are highly customizable and Jacobs has developed a number of proprietary SUMO process modules. The SUMO software also includes an influent characterization tool with advanced features for defining the fractionation of the organic components of the influent, which is important for developing a well-calibrated and predictive model. The Jacobs team began process modeling with Jacobs' proprietary tool, Replica Process, but switched to SUMO due to challenges with influent characterization and difficulties achieving model convergence in Replica Process.

2. WWTP Process Overview

The process steps are described in the next sections and the flow through the plant is depicted in Appendix B.

2.1 Headworks

The 8" influent force main from the collection system's main lift station enters the plant at the preliminary screening system consisting of an automatic vertical cylindrical fine screen with ¼" perforations. The screen is sized for an average flow of 0.72 mgd and a maximum flow of 1.0 mgd, and functions to remove larger debris from the flow prior to entering the Basin #1 anoxic zone. The screenings are ejected into a trash bin and landfilled. An operational concern for the screening process is corrosion due to high levels of hydrogen sulfide throughout the collection system. A section drawing of the fine screen is shown in Figure 2-1. From the fine screen, the screened raw sewage flows by gravity to the influent control structure. A coarse bar screen located at the influent control structure provides screening if the fine screen must be bypassed for maintenance.

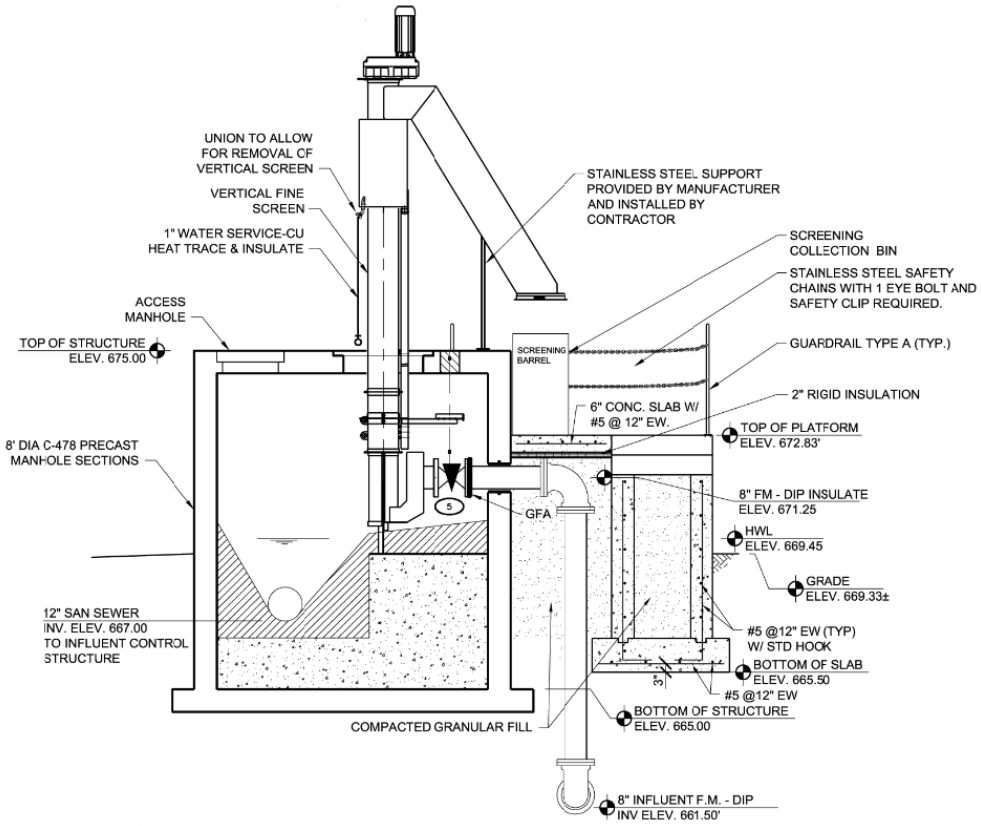

Figure 2-1. Section Drawing of Fine Screening Facility [1]

2.2 Primary Anoxic Zone

After passing through the influent control structure, the flow enters Basin #1 at the primary anoxic zone and is mixed with nitrified recycle, which is returned to this zone from cell 5 of the MBBR via the recycle pumps. This unaerated zone functions to begin the denitrification process and provide flow equalization prior to the moving bed biofilm reactor (MBBR). There are mixers within this zone to keep solids in suspension.

2.3 Moving Bed Biofilm Reactor (MBBR)

This secondary treatment process utilizes plastic media to support the attached growth of biofilm. The MBBR consists of six cells in series installed in the center of Basin #1 between the primary anoxic zone and the settling zone. Cells 1, 2, and 6 are anoxic and are equipped with both mixing and aeration equipment. Cells 3, 4, and 5 are aerobic and are equipped with only aeration equipment. Cell 5 is a swing zone, allowing it to operate as either anoxic or aerobic. The plant operations are presently operating cell 5 in an aerobic reactor mode. The air distribution system consists of two positive displacement blowers, 6" air header, valving for each MBBR cell, and the coarse air diffuser grid displayed in Figure 2-2. Operational challenges identified by the operational staff for the MBBR are as follows:

- Clogging of media retention screens
- Clogging of valves between cells
- Accessing valves and screens between cells for maintenance is difficult
- MBBR effluent weir increased the operating levels within Basin # 1, potentially contributing to the screening issues. This effluent weir was added during construction after the hydraulic testing of the WWTP to allow for the operation of the recycle pumps.
- The higher WSL of the MBBR resulted in more difficult maintenance of the mixers.
- It is assumed the higher WSL has impacted the operation of the scum boxes by allowing media to enter these boxes.
- Overflow of media from one cell to another resulting in deviations in the fill fractions away from the design recommendations.
- Air flow balancing between cells

The layout of the MBBR and downstream flocculation chamber and cavitation air flotation processes are shown in Figure 2-2.

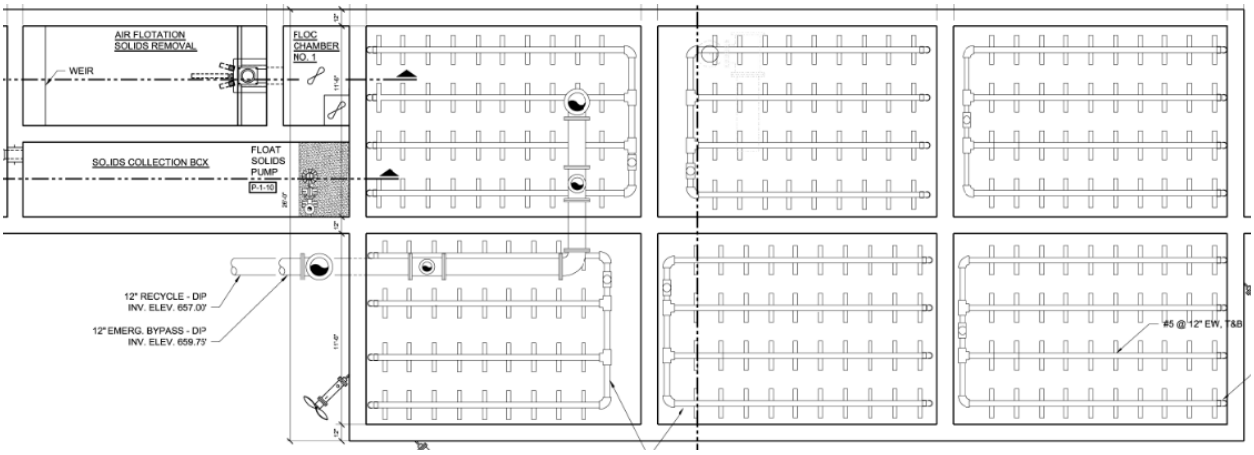

Figure 2-2. MBBR Lower Plan Drawing [1]

2.4 Cavitation Air Flotation (CAF)

After the MBBR Cells there is a CAF system that is intended to float lighter solids to the surface then remove them to the Biosolids Holding Basin via a skimming mechanism. Currently the CAF is being bypassed due to an out-of-service valve. Thus, the current flow path takes the MBBR Cells' effluent to a ferric dosing and flocculation zone and then into the Settling Zone of treatment Basin #1. Prior to the malfunction of the valve at the influent to the CAF, the system was inefficient and not performing as designed.

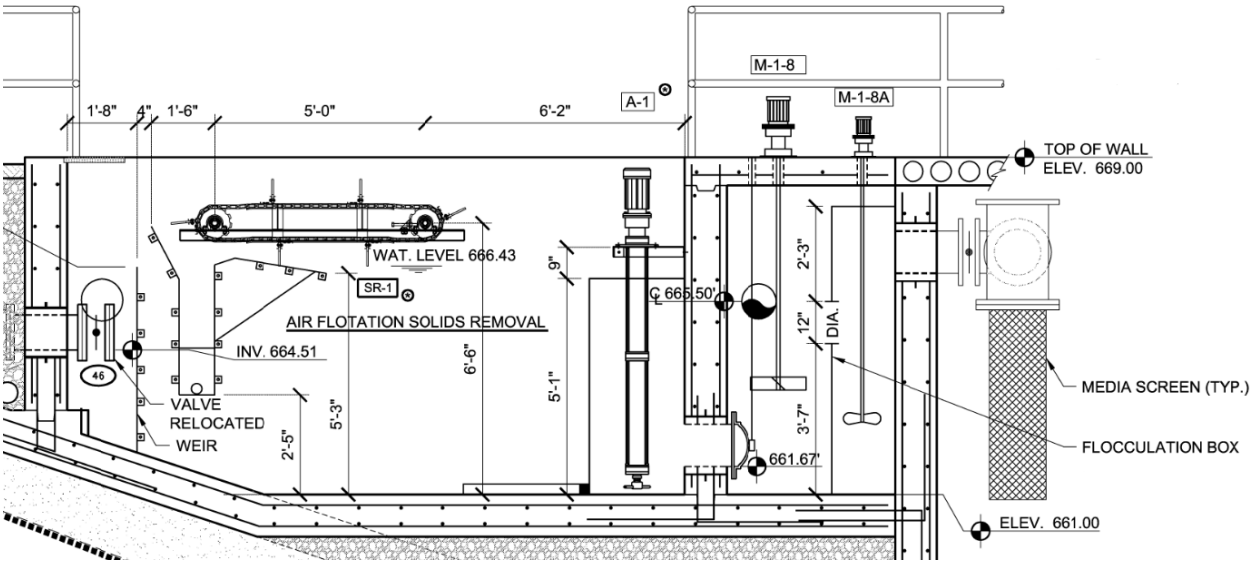

Figure 2-3. Section Drawing of Flocculation Zone and CAF Process [1]

2.5 Settling Zone

After the addition of ferric chloride in the flocculation zone, the waste stream flows into the settling zone of Basin #1 via pipe. The settling zone functions to allow solids to develop a blanket and for the liquid stream to be decanted to the polishing filters. The solids blanket in the settling zone is manually pumped

to the biosolids holding basin via a length of perforated solids withdrawal pipe at the deeper end of the settling zone along the outside wall of MBBR cells 4, 5, and 6. The solids withdrawal pipe is connected to the same recycle pumps that pump the nitrified recycle from MBBR cell 5 to the anoxic zone. However, due to deficiencies within the removal system most of the solids are removed during the annual cleaning of the settling zone. Operational concerns for the settling zone are listed below:

- Inefficient solids removal and transfer to the holding basin due to design.
- Thick solids build up within the zone due to low effectiveness of solids removal.
- Ballast sand has shifted over time down the slope of the zone to the center, the punctures caused by this were fixed, but the sand has not been reset.

Figure 2-4 shows a section view of the settling zone.

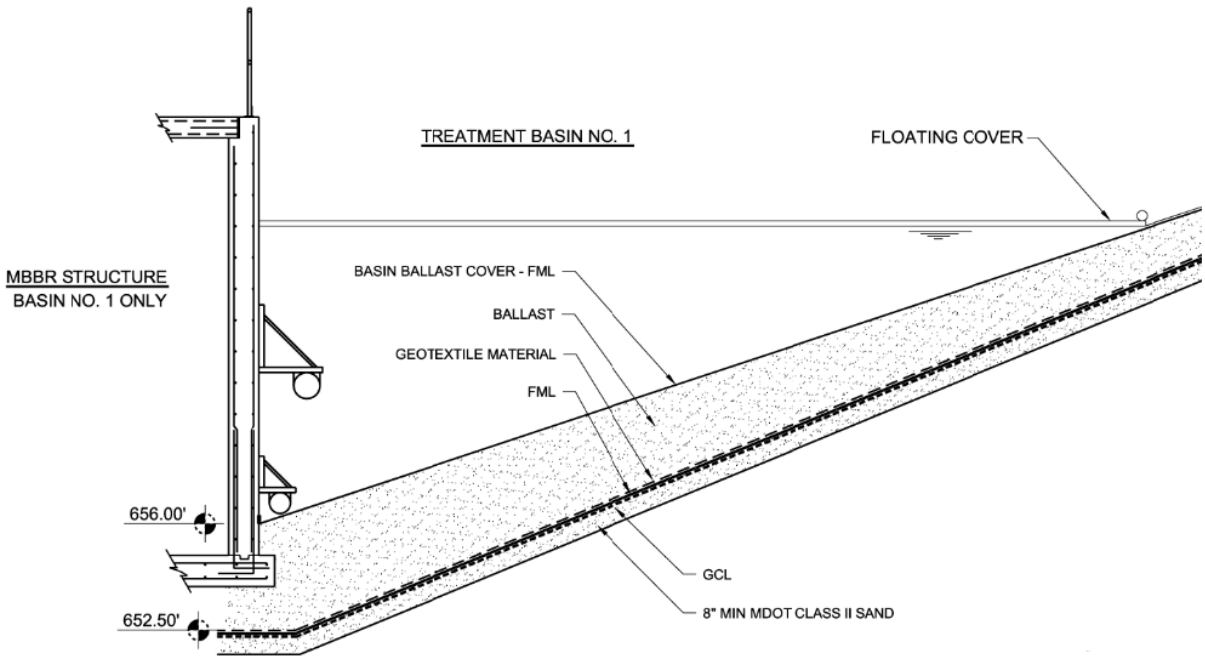

Figure 2-4. Section Drawing of the Settling Zone [1]

2.6 Polishing Filters

The Settling Zone is decanted into a pipe directing it to the polishing filters. These filters are continuous backwash, up flow sand filters that further remove solids and particulate phosphorus. The settling zone decant enters Filter no. 1 where it is evenly distributed at the bottom to the sand bed. The flow then travels upward through the sand bed during which the suspended solids are removed. The clean filtrate (effluent) passes over the filtrate weir at the top and leaves the unit. The filtrate from Filter No. 1 is fed to Filter No. 2 for additional polishing. The filters are designed for a peak flow of 100 gpm (6,000 gph or 144,000 gpd) with 30 to 100 mg/L of total suspended solids (TSS) [2]. Ferric chloride can be fed prior to the filters, but the plant does not dose at this location currently. The two filters are designed to be operated in series. Although the O&M Manual [2] states that the filters can be operated in parallel if needed, the two filters overflow the effluent weirs at different elevations and do not have equivalent

treatment capacity. Filter No. 1 has a media depth of 80-in while Filter No. 2 has a media depth of 40-in. Due to the different effluent weir elevations for each filter, flow will overflow from Filter No. 2 before the level in Filter No. 1 reaches the weir, resulting in most or all of the flow bypassing Filter No. 1. Due to the difference in depth and the hydraulic grade line, parallel operation is hydraulically infeasible. Attempting to operate the filters in parallel would result in operational issues such as unequal flow distribution and unbalanced solids removal. Cross sections of these filters are shown in Figure 2-5. Other common operational complications are listed below.

- Clogging of the filter reject (backwash) lines with no clean out to clear the blockages.
- Back up of flow due to the narrow diameter effluent flowmeter. Effluent flow meter is currently bypassed during summer months to prevent plant overflows.
- Minimal separation between the filter reject stream and the clean filtrate (effluent) leaving the filters, should either the effluent or reject back up due to a fault, one stream will flow into the other. This could increase effluent TSS and TP concentrations and potentially result in the facility not meeting effluent limits.

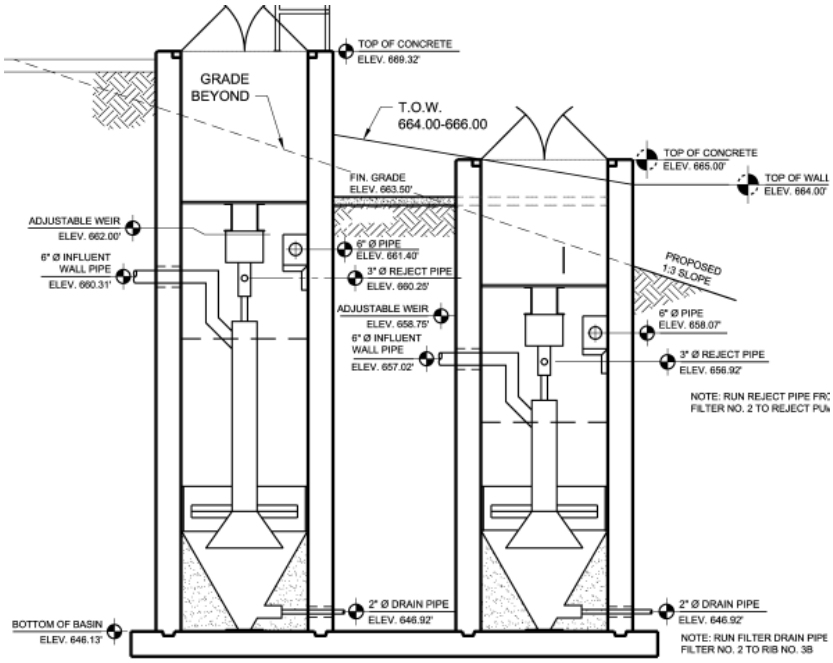

Figure 2-5. Section Drawing of Polishing Filters [1]

2.7 Effluent Flow Metering

Effluent flow is measured by a 3" magnetic flow meter located between the polishing filters and effluent control structures 2 and 3. Upstream of the flow metering structure, the 12" pipe carrying filtered effluent passes through a 12" x 8" x 6" reducing tee. The 6" branch reduces further to 3" before passing through the flow meter. The 8" branch has a gate valve and acts as a flow meter bypass when the flow rate is too high to pass through the 3" piping, which occurs at about 90 gpm (0.130 mgd). Downstream of the metering structure the 8" and 3" branches connect at another 12" x 8" by 6" tee and another section of 12" pipe carries the flow to effluent control structures 2 and 3.

2.8 Rapid Infiltration Beds (RIBs)

Once through the polishing filters, the liquid stream flows into an effluent control structure and to the selected RIB for ground water infiltration. A diffuser structure within the RIB disperses the effluent flow evenly over the bed surface. There are three RIBs, each with different permitted application rates, see Appendix A. To maintain the loadings under the permitted thresholds, effluent flow is rotated between the RIBs.

2.9 Biosolids Holding Basin

Solids wasted from the Settling Zone are pumped to a Holding Basin. The solids are held within the basin until they are removed for land application (typically every 5 years). The holding basin is decanted to thicken the solids over time and prevent overflow. The decant is recycled back to the influent control structure to combine with the influent flow to the primary anoxic zone. Operational concerns for the holding basin are:

- Per design a floating circulator was included to create an aerobic layer at the basin surface to help reduce odors. However it is currently out of service, resulting in no active surface mixing within the biosolids holding basin.
- Hauling during decanting can cause significant solids loading to enter the plant influent.
- There is no walkway accessing the center of the basin to allow sludge depth measurements or sampling.
- There is no sample port for measuring the flow from the settling zone in Basin #1 prior to entering the holding basin.

3. Current Wastewater Flows and Loads

This section presents the influent and effluent design basis for the project, which includes the following information:

- Historical flows, concentrations, and loads for parameters routinely measured at the WWTP
- Results of special sampling performed to characterize raw wastewater for accurate process simulation
- Selected design flows and loads
- Effluent requirements

3.1 Historical Data

The WWTP routinely collects data for influent flow, 5-day biochemical oxygen demand (BOD_5), total suspended solids (TSS), ammonia nitrogen (NH_3-N), and total phosphorus (TP). The plant typically uses a flow totalizer to record cumulative flow and then estimates an average daily flow by dividing the totalized flow by the number of days the flow was totalized. However, for this study raw flow meter data was obtained and analyzed to determine actual daily and hourly flow measurements. Grab samples are analyzed once per month for BOD_5, TSS, NH_3-N, and TP. Influent samples include a small stream of supernatant from Basin #2, where influent is redirected as needed to allow for maintenance.

Historical data from January 2021 to Dec 2024 was reviewed and summarized by season, with summer (busy-season) defined as from June through August and winter (off-season) as January through May and September through December. Table 3-1 presents the historical influent flows for these summer and winter periods. The seasonal nature of the influent flow rates can be observed in the time series chart in Figure 3-1.

Table 3-1. Historical Influent Flows

Season	Year	Average Flow (gpd)	Maximum Month Flow (gpd)	Maximum Day Flow (gpd)	Peak Hour (gph)
Summer	2020	84,584	89,974	114,792	8,817
	2021	82,606	91,930	136,994	10,654
	2022	68,915	73,456	87,623	9,456
	2023	65,951	73,365	83,817	6,349
	2024	72,170	77,956	99,845	6,747
Winter	2020	67,294	93,119	165,744	10,122
	2021	55,536	67,717	88,429	10,545
	2022	58,900	79,445	105,549	6,926
	2023	54,108	63,782	96,877	11,872
	2024	51,065	60,599	75,918	6,604

gpd = gallon(s) per day
gph = gallon(s) per hour

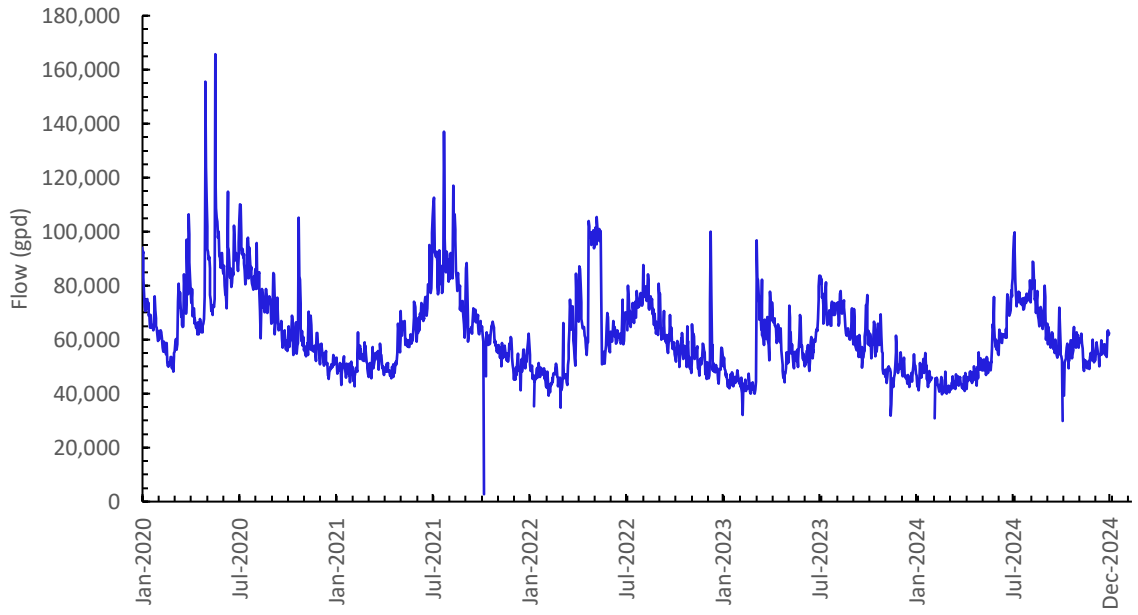

Figure 3-1. Historical Influent Flows

In addition, historical minute-by-minute flow data from January 2020 through February 2026 was evaluated to determine the peak hour flow which is also presented in Table 3-1. Peak hour flow is defined as the highest single-hour flow observed within a given year. While peak hour flow is not considered typical of system operation, it is a critical design consideration. Peak hour flows are important to consider in the sizing of each treatment process so that the plant is able to manage peak flows without causing overflows, filter media washouts, etc., which can cause damage to the plant, cause severe operational issues, and result in increased O&M costs. Table 3-2 summarizes the days the plant exceeded the existing polishing filters peak flow capacity of 6,000 gph (144,000 gpd) after June of 2023. There is limited and inconsistent evidence that the peak flow events coincide with same-day or day-prior rainfall. The capacity of the polishing filters is further discussed in detail in Section 5.2.3.

Table 3-2. Peak Flow Events (After June of 2023)

Date	Peak hourly flow (gph / gpd)	Duration (hours where filter capacity was exceeded)	Rainfall (inches) [a]	Notes
2023-07-03	6,284 / 150,822	2	0	
2023-07-04	6,043 / 145,027	1	0	
2023-07-05	6,349 / 152,386	1	0	
2023-07-15	6,011 / 144,255	1	0.02	
2023-08-17	6,016 / 144,383	1	0.98	
2023-08-18	6,121 / 146,904	1	0.02	
2023-09-29	7,795 / 187,069	1	0	
2024-05-27	6,604 / 158,505	2	0.29	
2024-06-30	6,010 / 144,233	1	0	There was 0.38-in rain the day before per NOAA

Date	Peak hourly flow (gph / gpd)	Duration (hours where filter capacity was exceeded)	Rainfall (inches) [a]	Notes
2024-07-03	6,351 / 152,433	2	0.53	
2024-07-04	6,166 / 147,985	2	0	
2024-07-05	6,710 / 161,050	3	0	
2024-07-06	6,747 / 161,922	1	0.03	
2024-07-07	6,096 / 146,308	1	0	
2024-07-18	6,144 / 147,457	1	0.05	
2024-08-04	6,442 / 154,610	1	0	
2024-08-09	6,419 / 154,066	1	0	
2024-08-10	6,574 / 157,780	3	0.24	
2024-08-11	6,506 / 156,155	1	0.07	
2024-09-03	6,517 / 156,398	1	0	
2025-03-20	6,056 / 145,353	1	0	Potential snow melt
2025-03-29	6,501 / 156,022	3	0	Potential snow melt
2025-03-30	7,486 / 179,661	8	0	Potential snow melt
2025-03-31	6,108 / 146,592	1	0	Potential snow melt
2025-05-13	6,670 / 160,068	1	0	
2025-05-25	6,184 / 148,422	1	0	
2025-06-21	6,382 / 153,179	1	0.27	
2025-07-01	6,007 / 144,158	1	0.03	There was 0.52-in rain the day before per NOAA
2025-07-02	6,032 / 144,775	1	0	
2025-07-03	6,381 / 153,138	2	0	
2025-07-04	7,276 / 174,621	8	0	
2025-07-05	7,337 / 176,078	7	0	
2025-07-06	7,448 / 178,762	6	0.86	
2025-07-07	6,274 / 150,565	1	0.09	
2025-07-12	6,080 / 145,925	1	0.49	
2025-07-20	6,171 / 148,101	1	0.36	
2025-07-24	6,370 / 152,876	1	0.42	

Notes:

[a] Rainfall data was acquired from National Oceanic & Atmospheric Administration (NOAA) Gauge Identifier: NORTHPORT 2 W, MI US (USC00206007)

Table 3-3 through Table 3-6 summarize the historical average concentration and loads for BOD_5, TSS, NH_3-N, and TP.

Table 3-3. Historical 5-Day Biochemical Oxygen Demand

Season	Year	Average Concentration (mg/L)	Average Load (lb/d)	Maximum Month Load (lb/d)
Summer	2021	101	63	91
	2022	151	83	106
	2023	150	74	119
	2024	213	123	160
Winter	2021	129	60	89
	2022	113	48	70
	2023	111	49	97
	2024	115	48	78

Table 3-4. Historical Total Suspended Solids

Season	Year	Average Concentration (mg/L)	Average Load (lb/d)	Maximum Month Load (lb/d)
Summer	2021	76	47	76
	2022	115	65	101
	2023	84	42	63
	2024	163	94	148
Winter	2021	74	35	60
	2022	79	34	56
	2023	70	32	78
	2024	67	28	49

Table 3-5. Historical Ammonia Nitrogen

Season	Year	Average Concentration (mg/L)	Average Load (lb/d)	Maximum Month Load (lb/d)
Summer	2021	24.8	16	21
	2022	34.6	19	23
	2023	39.3	19	32
	2024	38.8	22	25
Winter	2021	24.4	11	13
	2022	22.6	10	18
	2023	26.2	11	18
	2024	23.9	10	16

Table 3-6. Historical Total Phosphorus

Season	Year	Average Concentration (mg/L)	Average Load (lb/d)	Maximum Month Load (lb/d)
Summer	2021	3.6	2.3	3.0
	2022	4.2	2.3	3.3
	2023	4.7	2.3	4.1
	2024	6.2	3.6	3.9
Winter	2021	3.4	1.6	1.9
	2022	2.9	1.3	3.0
	2023	3.4	1.5	2.4
	2024	3.2	1.3	2.4

3.2 Influent Characterization

To increase the accuracy of treatment process simulation, special sampling and analyses were conducted to further characterize the raw wastewater. The special sampling campaign ran from September 15th to 29th,2025, and additional sampling was performed from November 18th to 20th, 2025. Table 3-7 summarizes the wastewater characterization parameters developed using special sampling data. During calibration of the SUMO model, some of the parameters were adjusted, and the adjusted values are also presented in the table. Model calibration is discussed further in Section 4.

Table 3-7. Wastewater Parameters Developed Using Special Sampling Data

Parameter	Special Sampling Value	SUMO Calibration Value
Alkalinity (as calcium carbonate)	388 mg/L	388 mg/L
COD/BOD_5[a]	1.27	2.2
Particulate BOD_5/BOD_5	0.51	0.51
Soluble COD/COD	0.56	0.56
Filtered flocculated COD/COD	Not measured	0.34
VFA/sCOD[b]	Not measured	0.09 (Summer) 0.12 (Winter)
VSS/TSS	0.92	0.92
NH_3-N/TKN	0.82	0.82
PO_4-P/TP[c]	0.71	0.68
Soluble nonbiodegradable COD/sCOD	0.17	0.17
Particulate nonbiodegradable COD/COD	0.11	0.11

[a] During SUMO process model calibration, the ratio was adjusted to 2.2 which aligns with typical value in US wastewater facilities. In addition, the COD results from special sampling campaign appeared to be unreliable.
[b] During SUMO process model calibration, the ratio was adjusted for summer and winter separately to close the phosphorus balance.
[c] During SUMO process model calibration, the ratio was adjusted to 0.68 to close the phosphorus balance.

Parameter	Special Sampling Value	SUMO Calibration Value

$cBOD_5$ = carbonaceous 5-day biochemical oxygen demand
BOD_5 = 5-day biochemical oxygen demand
COD = chemical oxygen demand
NH_3-N = ammonia nitrogen
TKN = total Kjeldahl nitrogen
TP = total phosphorus
VSS = volatile suspended solids

3.3 Influent Design Basis – Current Conditions

WWTP influent data from 2021 through 2024 were analyzed and used with the influent characterization to develop the influent design basis for the Northport WWTP. For each calendar year, the following parameters were identified:

- Flow-rate peaking factors (maximum month average) relative to annual average
- Flow-weighted average concentrations of TSS, BOD_5, NH_3-N, and TP
- Mass load peaking factors (maximum month average) for TSS, BOD_5, NH_3-N, and TP relative to annual average

Design flow-rate peaking factors, average concentrations, and mass load peaking factors were selected using a time-weighted averaging approach that placed greater weight on more recent years. TKN loads were calculated based on the ratios determined by the influent characterization. Table 3-8 presents the influent design basis for the WWTP. Maximum day design loads could not be determined from the historical data since BOD_5, TSS, NH_3-N, and TP were sampled only once per month.

Table 3-8. Northport WWTP Influent Design Basis – Current Flows and Loads

Parameter	Summer				Winter			
	Seasonal Average	Maximum Month	Maximum Day	Peak Hour	Seasonal Average	Maximum Month	Maximum Day	Peak Hour
Flow rate	72,080 gpd	78,508 gpd	98,457 gpd	187,291 gpd	55,121 gpd	68,048 gpd	94,143 gpd	132,170 gpd
BOD_5	170 mg/L				115 mg/L			
	103 lb/d	142 lb/d			53 lb/d	89 lb/d		
TSS	121 mg/L				71 mg/L			
	73 lb/d	113 lb/d			33 lb/d	63 lb/d		
NH_3-N	36.73 mg/L				24.35 mg/L			
	22.09 lb/d	28.75 lb/d			11.20 lb/d	17.71 lb/d		
TKN	44.95 mg/L				29.81 mg/L			
	27.04 lb/d	35.19 lb/d			13.71 lb/d	21.68 lb/d		
TP	5.08 mg/L				3.20 mg/L			
	3.06 lb/d	4.23 lb/d			1.47 lb/d	2.65 lb/d		

3.4 Effluent Design Criteria

The most recent permit for the Northport WWTP to discharge to the groundwater via rapid infiltration beds was set to expire on October 1, 2020 but has been extended and is pending renewal. The effluent limitations stated in the permit are summarized in Table 3-9. Jacobs operations staff have noted there may be future ammonia and total inorganic nitrogen (TIN) limits.

Table 3-9. Northport WWTP Effluent Requirements based on 132,000 gpd Daily Influent Flow

Parameter	Monthly	Minimum	Maximum
Flow	-	-	405,000 gpd
BOD_5	30 mg/L	-	45 mg/L
TP	0.3 mg/L	-	0.5 mg/L
pH	-	6.5 s.u.	9.0 s.u.

s.u. = standard unit(s)

3.5 Potential Future Regulations

The Northport WWTP's groundwater discharge is authorized under Rule 323.2218 of the State's Part 22 Groundwater Quality Rules, and discharge limits are subject to Rule 323.2222, which specifies an effluent TIN limit of 5 mg/L unless a more permissive effluent TIN limit can be shown to maintain down-gradient groundwater below 5 mg/L TIN. An effluent TIN limit as low as 5.0 mg/L may be included in a future discharge permit for Northport WWTP.

New or more stringent limits could reduce the influent pollutant loads the WWTP can treat while maintaining compliance, thereby impacting the capacity of the WWTP. As currently operated, the WWTP would not meet an effluent TIN limit of 5.0 mg/L. To meet a TIN limit, changes would be required to how the WWTP is operated and/or configured. Evaluation of meeting such a limit is beyond the scope of this evaluation. Therefore, Jacobs has based this capacity evaluation on the discharge limits in the current permit.

4. Model Development and Calibration

A steady-state whole-plant model was developed using SUMO, a commercial wastewater treatment process simulator, to evaluate treatment plant capacity.

To develop the model, Jacobs completed the following:

- Reviewed and analyzed historical data (Section 3.1) and influent characterization (Section 3.2) to develop influent flow and loadings (Section 3.3) for BOD_5, TSS, NH3-N, TKN, and TP for both winter and summer.
- Created a steady-state SUMO model using unit process-specific parameters (such as tank volumes, media fill fractions, etc.).
- Calibrated the model against the historical operation and performance data (for example, effluent quality, solids production, etc.).

Model calibration is an iterative process. The key operating parameters are adjusted up and down, until the model predicted values for key performance indicators match reasonably well with the plant data (that is, within the typical acceptable error range). The model calibration primarily focused on key parameters that affect the mass balance (that is, effluent BOD and TP, decant flows and concentrations).

The simulated conditions for calibration included the following:

- Current summer seasonal average flow, seasonal average loads, and seasonal average temperature
- Current summer maximum month flow, maximum month loads, and seasonal average summer temperature
- Current winter seasonal average flow, seasonal average loads, and seasonal average temperature
- Current winter maximum month flow, maximum month loads, and seasonal minimum month winter temperature

The model-predicted sludge production will be used to recommend alternative secondary settling technologies, to be addressed in detail within the Task 3 technical memorandum.

4.1 Modeling Assumptions

During the development of the model the following assumptions were made:

- Influent characterization from the special sampling campaign remains the same between summer and winter seasons, except the VFA to soluble COD ratio as noted in Table 3-7.
- Data available for the plant influent does not include plant recycles.
- MBBR cells 1, 2, and 6 are anoxic cells and the DO setpoints were set to 0. MBBR cells 3, 4, and 5 are aerobic cells and the DO setpoints were set to 4.0, 2.0, and 2.0, respectively.
- The biofilm and biofilm carrier parameters were set as follows:
    - Three biofilm layers were assumed
    - Biofilm thickness was assumed to be 60 micron
    - Boundary layer thickness was assumed to be 45 micron

  - Specific surface of biofilm carrier was calculated based on fill fraction of 50%, total biofilm surface area, and volume in each cell.
  - Bulk liquid volume displacement was set to 0.15.
- MBBR media retention screens are operational.
- The secondary settling zone was modeled as a three-compartment model for hydraulics and the reactions were made reactive.
- The secondary settling zone provides effective solids separation, thus allowing for the settling zone to be modeled as a functioning secondary clarifier.
- The solids holding basin operates as an anaerobic digestion lagoon that will result in volatile solids reduction over time.
- The biosolids holding basin was modeled as a generation 2 pond/lagoon in the model to simulate long-term behavior of the holding basin as the solids in the holding basin are hauled once every five years. The steady state depth of the sludge layer (sediment layer) was set to 5 ft and the TSS capture was set to 95%.
- Pumps operate at designed flow rates, per specification and operation manual.

4.2 Model Calibration

The SUMO model was calibrated for both the summer and winter seasonal average conditions, then the maximum month average loadings for each season were modeled to validate the model and evaluate current plant capacity in use. A multiplier was then applied iteratively to the maximum month winter and summer flows and loads to determine how much additional loading can be treated by the biological process. The results of each season's scenarios are discussed within the sections below.

4.2.1 Summer Season

The summer busy-season (June through August) was simulated for both current and maximum month flows and loads.

Under current seasonal conditions, the model predicted effluent concentrations below permit limits (Table 4-1). The simulator indicated higher BOD and ammonia-removal which was not fully consistent with observed plant performance. This may be due to additional biological processes occurring in the sludge blanket in the settling zone, which are not accounted for in the process model. The effluent TP concentrations closely matched the observed data. In addition, the biochemical transformations occurring in the unaerated, unmixed biosolids holding basin likely increase the BOD_5 and ammonia loads in the decant flow returned to secondary treatment, likely contributing to the higher ammonia and BOD_5 levels observed in the plant effluent compared to model predictions. Due to the limited solids data available for the secondary settled sludge, hauled sludge, and the decant, accurately simulating and matching current plant performance was challenging.

Table 4-1. Simulations Results of Current Summer Seasonal Average flow, Seasonal Average Loads, and Seasonal Average Temperature

Parameter	Units	Observed	Modeled Output	Difference (% or Absolute)	Notes
Effluent					
TP	mg/L	0.21	0.20	0.01	Within acceptable range
NH_3-N	mg/L	3.25	0.26	2.99	Possible ammonia release in settling zone, ammonia load from decant
BOD_5	mg/L	6.24	0.59	5.65	Possible increased BOD_5 in decant is increasing loads
TIN	mg/L	8.21	36.40	-28.19	Modeled plant is potentially carbon limited

4.2.2 Winter Season

The winter off-season (January through May and September through December) was simulated for both current and maximum month flows and loads.

Similar to the summer season, under current winter season conditions, the model predicted higher BOD and ammonia-removal compared to the observed plant performance. The discrepancies in the observed and modeled outputs can be attributed to the biological processes occurring in the secondary settling zone and the biosolids holding basin.

Table 4-2. Simulations Results of Current Winter Seasonal Average flow, Seasonal Average Loads, and Seasonal Average Temperature

Parameter	Units	Observed	Modeled Output	Difference (% or Absolute)	Notes
Effluent					
TP	mg/L	0.14	0.13	0.01	Within acceptable range
NH_3-N	mg/L	3.25	0.15	3.10	Possible ammonia release in settling zone, ammonia load from decant
BOD_5	mg/L	6.24	0.33	5.92	Possible increased BOD_5 in decant is increasing loads
TIN	mg/L	8.21	11.14	-2.93	Modeled plant is potentially carbon limited

5. Wastewater Treatment Plant Capacity Evaluation

5.1 Design Capacity

Northport WWTP was designed in 2008 to treat the flows and loadings presented below in Table 5-1. It is important to note that the plant is currently not operating as the original design intended due to operational concerns mentioned in the process overview section above (not all inclusive). Thus, the design capacity of the plant differs from the current operational capacity.

Table 5-1. Design Capacity [1]

Parameter	Value
Annual Average Daily Influent Flow	0.132 MGD
Minimum Daily Influent Flow	0.050 MGD
Maximum Daily Influent Flow	0.264 MGD
Peak Hour Influent Flow	0.488 MGD
BOD_5 Maximum Loading	456 lbs/d
TSS Maximum Loading	456 lbs/d
TP Maximum Loading	13 lbs/d
Ammonia Nitrogen, as N	57 lbs/d
TKN	77 lbs/d

5.2 Unit Process Capacities

5.2.1 Screening

The influent fine screen has a maximum hydraulic capacity of 1.0 mgd. The existing fine screen lacks redundancy but can be bypassed, with flow passing through the coarse bar screen. The screening process is not considered limiting for the WWTP's hydraulic capacity. However, bypassing the fine screen is not recommended due to the possibility of the MBBR media becoming clogged over time. The current main lift station pumps need to continue to be operated lower than their design 60 Hz to prevent hydraulic overloading of the plant, should the hydraulic loading increase to allow for the full 1,350 gpm design point, the Hz can be increased on the pump. Jacobs recommends expanding the fine screening facility to include 2 units, one duty and one standby, each with capacity to handle the design peak hour flow.

5.2.2 Secondary Treatment

The secondary treatment process includes the biological process (pre-anoxic zone and MBBR), metal salt addition, the CAF, and the settling zone.

5.2.2.1 Biological Process

The Sumo model was used to confirm the capacity of the biological process. The summer maximum-month simulations produced results similar to the observed seasonal average simulation for effluent

BOD_5, ammonia, and TP, as discussed in Section 4.2.1. Table 5-2 summarizes the current summer maximum month flows and loads across the unit processes and estimated remaining capacity.

Table 5-2. Simulation Results for Current Summer Seasonal Maximum Month Flow, Seasonal Maximum Month Loads, and Seasonal Average Temperature

Parameter	Units	Modeled Output	Unit Design Capacity per O&M	Estimated Remaining Capacity	Limiting Unit
Influent					
Flow	gpd	78,602			
TSS	lbs/d	128.96			
TP	lbs/d	4.26			
NH_3-N	lbs/d	29.72			
BOD_5	lbs/d	146.00			
Decant					
Flow	gpd	31,162	1,44,000	78%	Decant Pump
TSS	lb/d	23.88			
TP	lb/d	0.88			
NH_3-N	lb/d	0.08			
BOD	lb/d	0.21			
MBBR					
Air Flow	scfm	180.62	410 at max. month; 819 at max. day	56% at max. month; 78% at max. day	Blower
Flow	gpd	109,784			
TSS	lb/d	179.52			
TP	lb/d	5.31			
NH_3-N	lb/d	0.29			
BOD_5	lb/d	25.12			
Nitrified Recycle Flow	gpd	360,000	360,000	0%	Pump
Secondary Settling Zone					
Ferric Feed Flow	gpd	20	72	72%	Pump
Secondary Effluent Flow	gpd	107,427			
Secondary Effluent TSS	lbs/d	26.00			
Secondary Waste Sludge Flow	gpd	2,357			
Reject Flow	gpd	28,800	43,200	33%	Pump
Reject TSS	lbs/d	19.94			

Parameter	Units	Modeled Output	Unit Design Capacity per O&M	Estimated Remaining Capacity	Limiting Unit
Effluent					
Flow	gpd	78,627	405,000		Permit
TSS	mg/L	9			
TP	mg/L	0.35	0.3		Permit
NH_3-N	mg/L	0.45			
BOD_5	mg/L	1.30	30		Permit

For the capacity evaluation, the flows and loads were increased iteratively to assess the MBBR treatment capacity. Based on the simulation results (Table 5-3), it can be concluded that the MBBR can treat a 2.5-fold increase and meet the current permit BOD_5 limits. The simulator indicates that the current TP limits can be met; however, compliance is largely constrained by the polishing filters' capacity. At increased loads, after ferric addition most effluent TP is present in particulate form, requiring optimal filtration to remove.

The air flow requirements indicate that the MBBR treatment capacity is primarily limited by the existing aeration blowers. Although the maximum day simulation was not completed due to insufficient historical data to characterize maximum day loading, maximum day air requirements are typically double the maximum month requirements.

Table 5-3. Simulation Results of 2.5x Summer Maximum Month Flows and Loads

Parameter	Units	Modeled Output	Current Permit Limit/Capacity	Remaining Capacity
Flow	gpd	196,290		
TSS	mg/L	5.00		
TP	mg/L	0.26	0.3	
NH_3-N	mg/L	0.49		
BOD_5	mg/L	1.56	30	
Air Flow	scfm	441	410	-8%

For completeness, the 2.5x winter maximum month flows and loads results are included in Table 5-4, however, the winter loads do not impact the capacity assessment as the loads are lower in the winter compared to summer.

Table 5-4. Simulation Results of 2.5x Winter Maximum Month Flows and Loads

Parameter	Units	Modeled Output	Current Permit Limit/Capacity	Remaining Capacity
Flow	gpd	170,106		
TSS	mg/L	8.45		
TP	mg/L	0.33		
NH_3-N	mg/L	0.98		
BOD_5	mg/L	2.16		
Air Flow	scfm	168	410	59%

In general, process simulations indicate the biological treatment process can accommodate up to a 2.5-fold increase in influent pollutant loads. However, improvements to secondary treatment solids separation are required for the overall secondary treatment process to meet this potential.

5.2.2.2 CAF

The CAF was designed to remove 166 lb/day of lighter solids downstream of the MBBR. The CAF is not currently operating and historically has not operated as designed. The capacity of the CAF was not determined as part of Task 2 but will be evaluated as part of Task 3.

5.2.2.3 Settling Zone

The settling zone was designed to remove 41 lb/day of heavier solids [2]. Due to the CAF not performing as designed, the solids loading on the settling zone is higher than the design intended, and much of that solids load is lighter and less easily settled compared to the design solids loading. The settling zone lacks an effective mechanism for sludge collection and removal and does not remove solids evenly from across the floor of the basin. The result is that a large portion of the generated solids remain in the settling zone until the annual cleaning. Additionally, the design of the settling zone is atypical, and no standard design criteria exist.

Since Jacobs modeled the settling zone as a secondary clarifier, biological processes that occur in the sludge blanket are not accounted for in the process simulations. In particular, ammonia release is likely to occur in this zone. The process model was not calibrated for nitrogen balance since the WWTP does not currently have ammonia or TIN limits, but without improvements, biological processes in the settling zone will likely limit the WWTP's ability to meet future nitrogen discharge limits.

Although Jacobs does not have a determination for the capacity of the settling zone at this time, it is our opinion that the settling capacity is more limiting of the WWTP's overall capacity than the MBBR, and the influent loads to the WWTP should not be increased without improvements to secondary treatment solids separation. A potential short-term solution would be more frequent cleaning to remove accumulated solids. Jacobs will evaluate secondary treatment solids separation and provide recommendations for improvements as part of Task 3, which will be reported in a separate technical memorandum.

5.2.3 Polishing Filters

The continuous-backwash upflow sand filters have an average design capacity of 132,000 gpd with influent TSS of 70 mg/L, and a design peak flow capacity of 100 gpm (that is 6,000 gph or 144,000 gpd)

[2]. Current peak hour flows (discussed in Section 3.1) already exceed this capacity. Any flow that bypasses the filters will not meet effluent TP limits, so the filtration process's hydraulic capacity must match the design peak hour flow.

Historical performance data also indicate that the filters are at times not meeting the design performance requirement to maintain effluent TSS below 5.0 mg/L. This performance variability increases the risk of exceeding the discharge limit for total phosphorus.

With a peak flow capacity of 6,000 gph (144,000 gpd), the polishing filters are undersized relative to the WWTP's design peak hour flow of 488,000 gpd (Table 5-1), and maximum day and peak hour flows recorded in the historical data have also already exceeded the peak flow capacity of the filters. Based on this, the WWTP has already exceeded its hydraulic capacity under existing conditions. The polishing filters represent the hydraulic bottleneck for the plant and currently are the limiting process for the plant's overall hydraulic capacity. Under the existing conditions, the WWTP cannot accommodate an increase in plant influent flows without operational changes or capital upgrades to the filtration process.

5.2.4 Effluent Flow Metering

Effluent flow metering currently must be bypassed at flows slightly higher than the current average flow due to the small diameter of the piping needed to accurately measure flow during low flow conditions. Improvements to effluent flow metering are required to handle even the current plant flows, although flow metering was not considered limiting for the purposes of this evaluation since higher flows can bypass the 3" magnetic flow meter through an 8" pipe.

5.2.5 Biosolids Storage and Hauling

The biosolids holding basin has adequate capacity to serve the plant as influent loads increase. However, the frequency of hauling will need to increase to accommodate higher solids production associated with higher influent loads. The solids holding basin operations could benefit with improvements to solids mixing and decant which will be further evaluated and discussed in a separate technical memorandum under Task 3.

6. WWTP Conclusions and Recommendations

6.1 Conclusions

The major findings of the capacity evaluation are as follows:

- The plant capacity is hydraulically limited by the polishing filters, which can only treat a peak hour flow of 6,000 gph (144,000 gpd). Peak hour flows from 2020 through 2024 ranged from 6,747 gph to 11,872 gph. Based on this, the WWTP is at times already exceeding its peak hydraulic capacity.
- The polishing filters do not consistently meet design effluent TSS requirements, putting the WWTP at risk of violating effluent TP limits because of the proportion of TSS that is particulate phosphorus.
- The biological treatment process at the plant, including the pre-anoxic zone and MBBR, can handle up to a 2.5-fold increase in influent loads.
- Secondary treatment solids separation processes, including the CAF and secondary settling zone, are likely already overloaded due to failure of the CAF to perform as designed, resulting in a higher load of lighter solids in the settling zone. The settling zone has poor collection and removal of settled solids and carries a blanket of floating solids. Secondary treatment solids separation will be further evaluated in a separate technical memorandum under Task 3.
- An effluent TIN limit as low as 5 mg/L is likely to be added to the WWTP's discharge permit at some time in the future. Historical data indicates that this limit would not be met at the current conditions. Evaluating operational and process configuration changes that would be needed to meet such a limit is beyond the scope of this evaluation.

6.2 Recommendations

For the plant to treat additional flows and loads, the hydraulic limitations of the polishing filters must be addressed. Jacobs recommends an expansion of filtration capacity to accommodate the peak hour flows associated with NLTUA's desired WWTP capacity. As an interim solution, the impact of peak hour flows could be mitigated by modifying existing structures to allow for automated hydraulic equalization with the use of Treatment Basin #2. The Task 5 Executive Summary will discuss both the short-term hydraulic equalization approach and long-term filter capacity expansion.

Jacobs recommends improvements to secondary treatment solids separation in order for the plant to treat additional influent pollutant loads. An evaluation of these processes and recommended improvements will be carried out as part of Task 3 and results and recommendations will be reported in a separate technical memorandum.

Additional recommendations include:

- Add redundancy for influent fine screening and address ventilation and hydrogen sulfide corrosion concerns
- Repair valves in the MBBR
- Adjust elevations of media screens and scum boxes in the MBBR to work with current operating levels that were adjusted higher than the design operating level during commissioning of the WWTP
- Add cleanouts to polishing filter reject lines
- Increase the hydraulic capacity of effluent flow metering

Part 2: Collection System

7. Introduction

The collection system capacity evaluation entailed using available data provided by NLTUA to develop a model of the collection system using PCSWMM, a storm management modeling software, and using the model to identify capacity issues within the system for various conditions. This document discusses development of the model and the evaluation of system capacity. The model will also be used to evaluate the system's capacity to handle projected population growth and commercial development in the Village in a future project phase.

7.1 Background

The sewer collection system is made up of 7.3 miles of gravity main, 4.8 miles of low-pressure main, and 1.0 mile of high pressure forcemain to deliver wastewater from the main pump station to the WWTP. The system has two other smaller capacity pump stations, one on 7th Steet and one on Northport Point Road (NPP), and approximately 155 residential grinder pumps to boost flow in the low-pressure main portion of the system. Each of the three pump stations have two pumps which operate in a lead-lag configuration. Figure 7-1 provides an overview of the collection system.

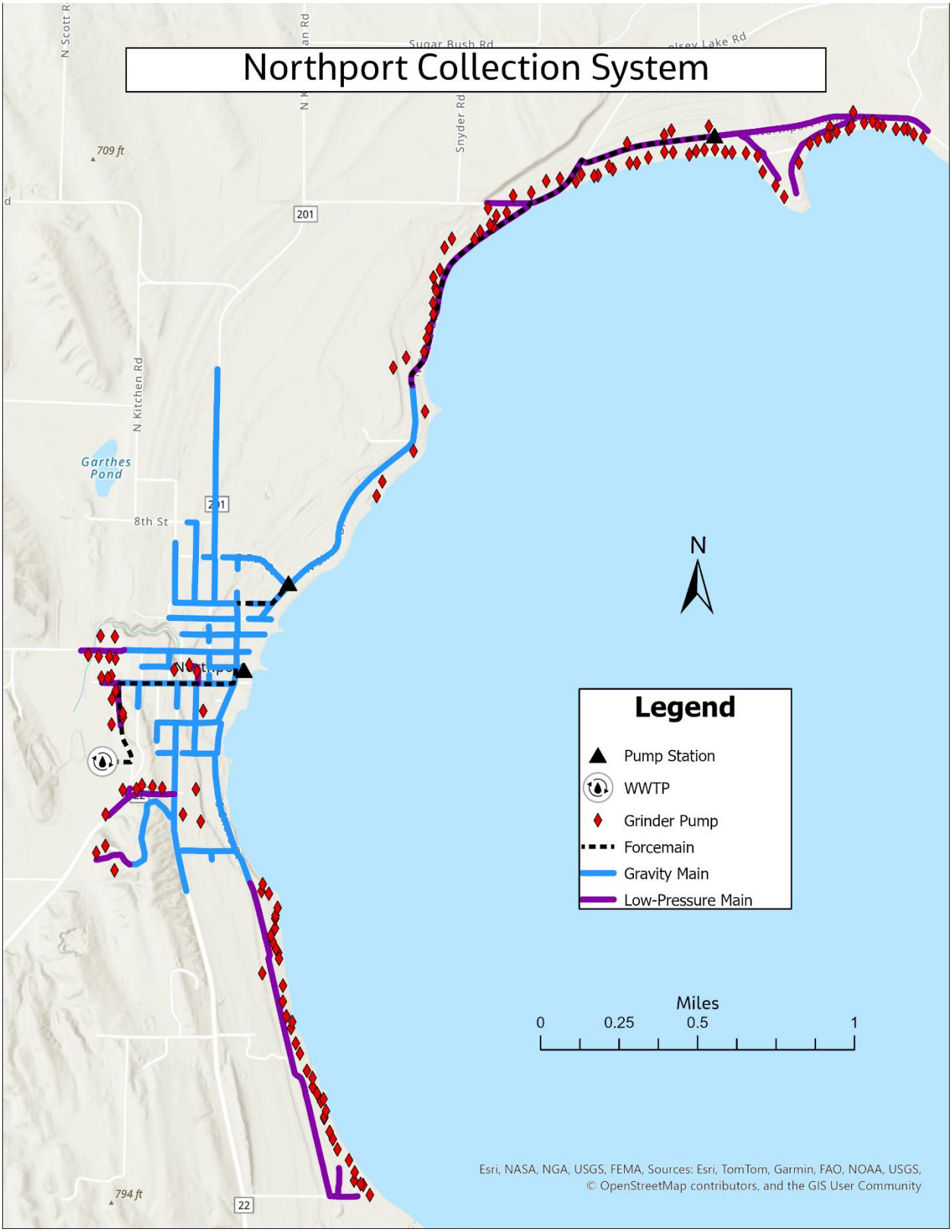

Figure 7-1. Map of Key Components of the Collection System

7.2 Scope of Work

The NLTUA retained Jacobs to perform capacity evaluation of the Northport collection system. The scope of the task includes:

- Prepare hydraulic model in PCSWMM software
- Determine hydraulic capacity of key segments of the system
- Identify potential limitations in pipe segments and pump stations

7.3 Capacity Evaluation Approach

The following summarizes the approach taken to develop the model and perform the capacity evaluation of the Northport collection system:

- Reviewed and summarized historical WWTP influent data to quantify summer and winter flows
- Developed PCSWMM model of the collection system using provided geodata, record drawings, and pump operational data
- Performed collection system capacity analysis using the model to identify areas with existing capacity issues

8. Existing System Data Review

This section summarizes the data used to develop the collection system model and conduct the system capacity evaluation. Critical data was provided to Jacobs when the project kicked off and was requested from NLTUA as needed as part of Task 1. The following list summarizes the data received for development of the collection system model:

- WWTP Influent Flow Data: Access was provided to Hach WIMS where the Northport WWTP data is stored. Daily influent flows from January 2024 through November 2025 were collected to characterize flows in the model.
- Sewer Collection System GIS Data: Data was used to create the collection system model network.
- Parcel GIS Data: Data was used to quantify system tributary area and inform flow allocation in the model.
- WWTP and collection system engineering prints:
    - WWTP prints were used to identify the collection system boundary condition at the WWTP, represented as an outfall in the model. This is the point at which wastewater exits the collection system and enters the treatment plant. This was used to calculate the lift from the main pump station to the WWTP [1].
    - Collection system prints were used to backcheck GIS data and provide information if GIS data was not available or was questionable. Pump station detail drawings were used to assign pump start/stop elevations for lead and lag pump operation [3].
- Pump manufacturer model numbers: Were provided by NLTUA for each of the three pump stations in the collection system. This information was used to calibrate the model with the respective performance curves to simulate existing conditions.

More details on data processing required for the model development and evaluation are included in subsequent sections.

9. Collection System Model Development

A model of Northport's collection system was developed using PCSWMM, an industry recognized storm management modeling software. The model was built in PCSWMM Version 7.7.3920 using SWMM engine Version 5.2.4. Figure 9-1 shows the collection system model and PCSWMM interface.

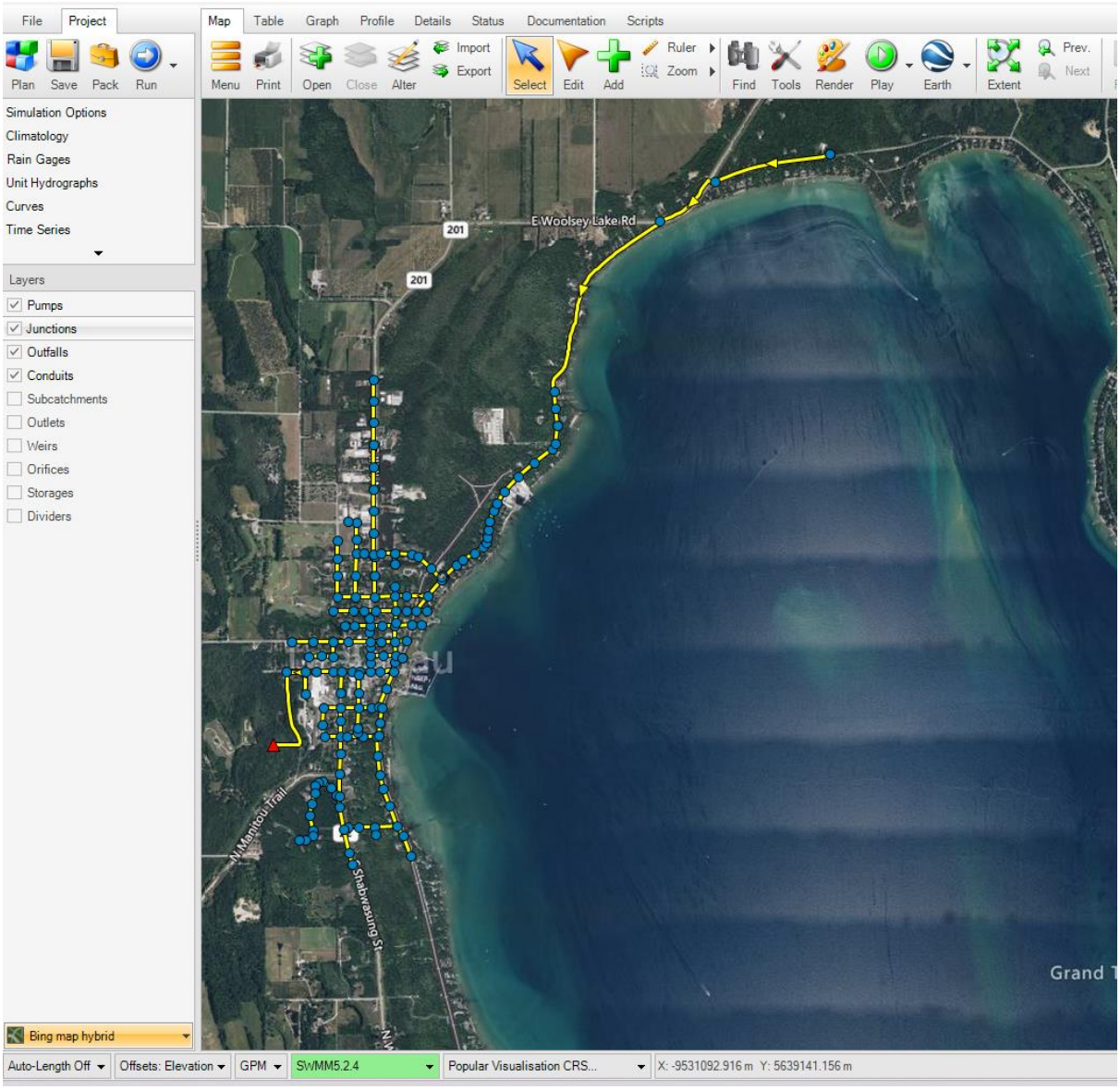

Figure 9-1. PCSWMM Interface.

9.1 Flow Characterization

Flow throughout the system varies seasonally due to increased home occupancy and tourism to the village in the summer. Accordingly, the model was evaluated for both summer (busy-season) and winter (off-season) to characterize system capacity throughout the year. For the evaluation, summer was defined as June through August, and winter was defined as September through May of the following year.

Daily influent flow to the WWTP from January 2024 through November 2025, collected from Hach WIMS, was used to characterize flow throughout the wastewater collection system and develop flow inputs for the model. Table 9-1. WWTP Influent Flow (January 2024 - November 2025) summarizes the WWTP influent flows for both seasons for the range of data reviewed. The minimum influent flow for each season was assumed to be the base sanitary flow, or the municipal wastewater contribution from residents and visitors. The difference between the average and minimum influent flow in each season was assumed to be the groundwater infiltration (GWI) to the sewer system.

Table 9-1. WWTP Influent Flow (January 2024 - November 2025)

Season	Minimum (gpd)	Average (gpd)	Maximum (gpd)
Summer	47,840	70,591	110,680
Winter	27,302	53,548	103,680

Flow resulting from rainfall derived inflow and infiltration (RDII) was calculated as the daily flow minus the average flow for the respective season for days when there was recorded rainfall and the daily rainfall exceeded the average flow. For these days the R-value, or the percentage of rainfall that enters the sewer system, was calculated based on the I/I flow volume, the daily rainfall depth, and the system tributary area. Table 9-2 summarizes the calculated R-values for both seasons. The R-value is ultimately used in the model to generate RDII based on the rainfall input.

Table 9-2. Percentage of Rainfall Volume That Becomes RDII (R-Value)

Minimum	Median	Average	Maximum
0.0000	**0.0024**	**0.0058**	0.0561

9.2 Model Development

9.2.1 Model Network Development

The NLTUA provided Jacobs with a geodatabase (.gdb) containing spatial information of the collection system components. These data included the gravity sewer lines, force mains, low-pressure mains, manholes, pump stations, and grinder pump locations. The GIS information also included attributes pertinent to model development, such as diameter, length, and invert elevations for the pipes and invert and rim elevations for manholes. These data were imported to the model to develop the model network. Collection system prints were then used to confirm the GIS attribute data imported to the model and to fill in data gaps.

Grinder pump locations were provided in GIS, but head and operational information were not available, so the grinder pumps were not modeled. Accordingly, the low-pressure force main downstream of the grinder pumps were also not included in the model because without the presence of the grinder pumps, the flow entering the system in these areas would pool at the load points due to the against-grade slope

of the low-pressure force mains. Instead, the flow for these areas was loaded in the model at the closest downstream location that was included in the model network.

9.2.2 Flow Allocation

According to GIS data, there are 737 service lines across the Northport gravity and low-pressure main system. In the absence of the availability of flow data throughout the system and water usage data, each lateral was assumed to have an equal flow contribution. The approximate daily average flow for each lateral connection was determined for both seasons by dividing the total daily flow assumed for base sanitary flow and GWI by the number of service laterals. To distribute flow throughout the system, the average flow for a lateral was multiplied by the number of laterals assigned to each manhole. The laterals were assigned to the manholes using the "Near" function in ArcGIS Pro and were adjusted after review in the PCSWMM model if needed.

RDII was applied to the model based on tributary area acreage. The parcels served by the collection system were grouped and assigned to the manholes similarly to the approach used to assign the laterals to the manholes. The sum of the acreage of the parcels assigned to each manhole was applied to the model manholes as the sewershed area to generate RDII within the model. An RTK unit hydrograph was applied to the sewershed area to generate RDII based on the rainfall for the wet weather scenarios evaluated. Table 9-3 shows the unit hydrograph values applied in the model where R = fraction of rainfall that becomes infiltration and inflow, T = time to hydrograph peak (hours), and K = falling limb / rising limb duration. R values were determined using precipitation data local to Northport [4], T and K values were selected through Jacobs standard practices. As seen above, the average R-value based on the WWTP daily influent flow data was approximately 0.006 (0.6%). Conservatively, the total R-value applied in the model was 1% and was divided evenly between the short-term, medium-term, and long-term response unit hydrographs. The T and K values used in the model are reflective of values within the general industry-accepted ranges for those values.

Table 9-3. Unit Hydrograph Used in Wet Weather Scenarios

Response	R	T	K
Short-Term	0.0033	0.5	1
Medium-Term	0.0033	3	3
Long-Term	0.0033	5	5

9.2.3 Assumptions and Data Limitations

The model was developed using assumptions based on data available at the time of model development and analysis. If additional data is provided, the model can be updated and re-evaluated in future phases of the project. Assumptions and data limitations are listed below.

- WWTP influent flow data was used to characterize the wastewater flows throughout the system. Data was retrieved from Hach WIMS which documents WWTP influent on a totalizer basis which is used to create a daily influent average over the days since the totalizer was last reset. Due to the absence of available flow data collected throughout the system and water usage data, flow was applied evenly throughout the system based on the number and location of laterals and parcels. This generalizes the flows throughout the system, and makes it difficult to identify potential GWI and RDII hot spots throughout the system.

- Pump runtime data for the three pump stations were provided, but were ultimately not used for development of the model. The pump runtime data were used to calculate daily pumping volume by multiplying the recorded daily pump operating hours with the assumed pump capacity. The calculated daily volume pumped conflicted with the WWTP influent data provided which was considered the primary source.
- Pump manufacturer model numbers were provided by NLTUA for each of the three pump stations. Pumps were assumed to operate along their respective pump performance curve which were either provided by the NLTUA or found online according to the model number.
- Pumps were assumed to operate according to the elevations specified in the pump station details in the collection system prints. In some cases, the listed elevations were crossed out but not replaced by another value. If an elevation was crossed-out in the prints but not replaced with another value, the crossed-out value was used.
- Manhole data provided in GIS did not include invert elevations. Manhole inverts were assigned based on connecting pipe segments invert elevations, and then back-checked using the plan and profile drawings from the collection system prints. In some cases, data sources conflicted and minor adjustments were made to resolve issues preventing the model from running correctly.
- Each lateral documented in the provided GIS data was assumed to have an equal daily wastewater contribution into the collection system since individual contributions were not available.
- The hydrograph applied to the wet weather events utilized Jacobs standard values and local historical rainfall data since flow monitoring data throughout the system was not available. Historical rainfall data local to Northport was obtained from the National Oceanic Atmospheric Administration's (NOAA) website [4].
- Grinder pump information (flow, head, and operational information) were not available, so the grinder pumps were not modeled. Accordingly, the low-pressure force main downstream of the grinder pumps were also not modeled because the flow entering the system in these areas would pool at the load points due to the against-grade slope of the low-pressure force mains. Instead, the flow for these areas was loaded in the model at the closest downstream location that was included in the model network.

10. Collection System Capacity Evaluation

10.1 Collection System Level of Service

The model was evaluated for dry weather flow and wet weather flow conditions. Information on an established design level of service (LOS) was not available for the collection system, so system capacity was evaluated for both a 2-year and 5-year LOS. These storm recurrence intervals are often used as LOS for sanitary sewer systems to evaluate system capacity. Hyetographs for the 2-year, 24-hour and 5-year, 24-hour storm events were developed using the SCS Type II hyetograph shape and rainfall volumes based on NOAA Atlas 14 Point Precipitation Frequency Estimates for Northport, Michigan (Station Name: NORTHPORT 2W) [5]. The corresponding total depth of rainfall for the 2-year, 24-hour and 5-year, 24-hour rainfall events are 2.26 inches and 2.76 inches, respectively. Figure 10-1 shows the resulting 2-year, 24-hour and the 5-year, 24-hour rainfall hyetographs in inches per hour (in/hr).

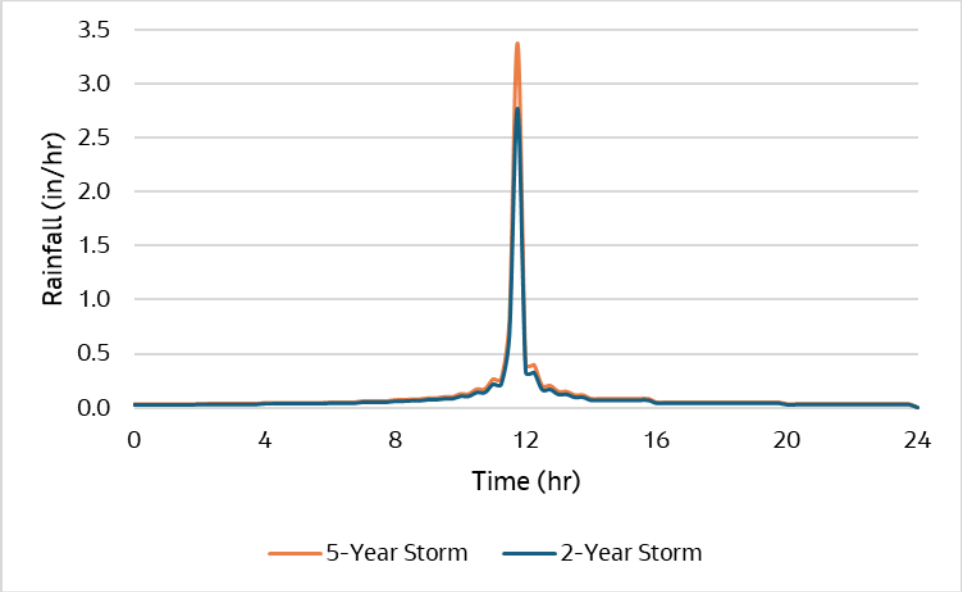

Figure 10-1. 2-Year, 24-Hour and 5-Year, 24-Hour Design Storm Hyetographs

10.2 Capacity Analysis Scenarios

The model was evaluated for six different scenarios:

1. Summer Dry Weather
2. Summer 2-Year, 24-Hour Storm Event
3. Summer 5-Year, 24-Hour Storm Event
4. Winter Dry Weather
5. Winter 2-Year, 24-Hour Storm Event
6. Winter 5-Year, 24-Hour Storm Event

The results for each scenario were reviewed to identify capacity issues throughout the system. Generally, the results for both summer and winter conditions are very similar, with the summer trending slightly worse with respect to capacity concerns. Maps showing the summer condition results are included in the body of the report and maps showing winter condition results are included in Appendix C. While Figure 7-1, above, provides an overview map of the entire collection system, the results maps below are zoomed in on the portion of the system included in the model to highlight the results for each scenario. Pump station operations were evaluated for the summer dry weather flow, 2-year, 24-hour, and 5-year, 24-hour storm scenarios.

10.3 Summer Dry Weather Results

The map of dry weather results shown in Figure 10-2 reveals that pipe flow through all segments included in the model is within 0 to 25% of the calculated pipe capacity. Additionally, no surface flooding or basement backup risk was found. There are no concerns with pump capacity for dry weather flow conditions. This indicates that Northport's collection system does not exhibit capacity issues during dry weather flow conditions.

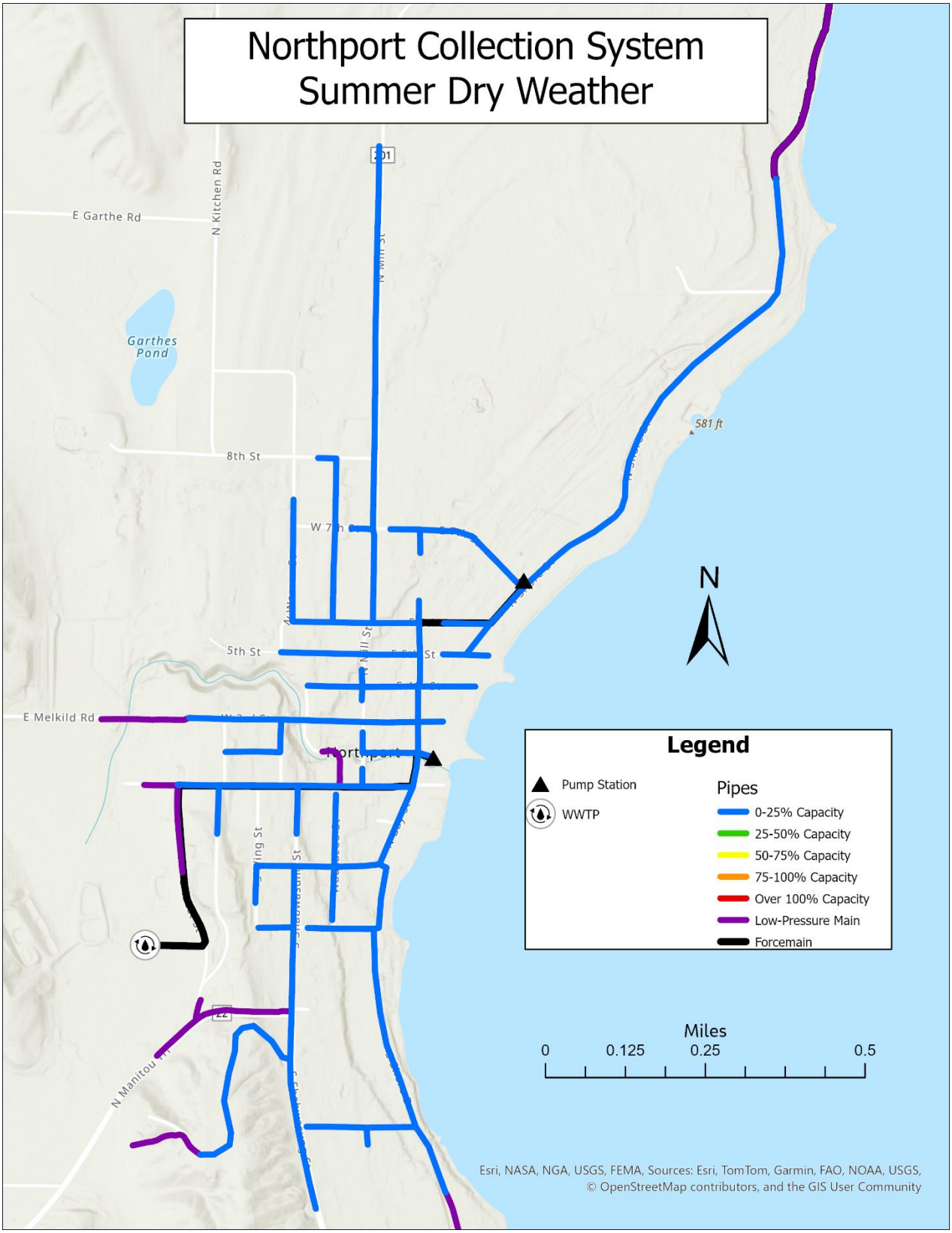

Figure 10-2. Summer Dry Weather Results

10.4 Summer 2-Year, 24-Hour Storm Event Results

Table 10-1 shows the model-predicted maximum inflow and maximum pumping rate of the three pump stations (PS) during the 2-year, 24-hour storm event, along with the capacity of each PS. The total pump station capacity was calculated as the manufacturer's rated pump flow, or design flow, multiplied by two (assuming two pumps running simultaneously). However, if multiple pumps run simultaneously, often the pumps won't operate at the rated or design flow.

The modeled maximum inflow at PS02 and PS03 exceed the pump station capacity for the 2-year, 24-hour storm event, indicating that the respective pump stations are undersized to convey the flow getting to the pump station for the respective storm event. The modeled maximum inflow at PS01 does not exceed the capacity of the pump station; however, the modeled maximum pumping rate is lower than the maximum total inflow, indicating that the modeled pumps are not operating at the design condition. This results in surcharge upstream of the pump station, which can cause potential basement backups and flooding manholes.

Table 10-1. Summer 2-Year, 24-Hour Storm Pump Station Performance

Pump Station Name	Pump Station Rated Capacity (gpm)	Maximum Total Inflow (gpm)	Maximum Pumping Rate (gpm)
PS01 (Main)	2,700	1,146	884
PS02 (7th Street)	390	742	268
PS03 (NPP)	100	132	99

The map of the 2-year, 24-hour storm event results is shown in Figure 10-3. Capacity issues are exhibited along North Mill Street, North and South Shore Drive, North Bay Street, and South Shabwasung Street. Houses along these streets are at risk of basement backups. Two manholes, one at the intersection of North Shore Drive and 5th Street and one at the intersection of North Bay Street and Main Street, are projected to have surface flooding during a 2-year, 24-hour storm event. Thirty-two (32) pipe segments are shown to operate beyond their calculated flow capacity during the 2-year, 24-hour storm.

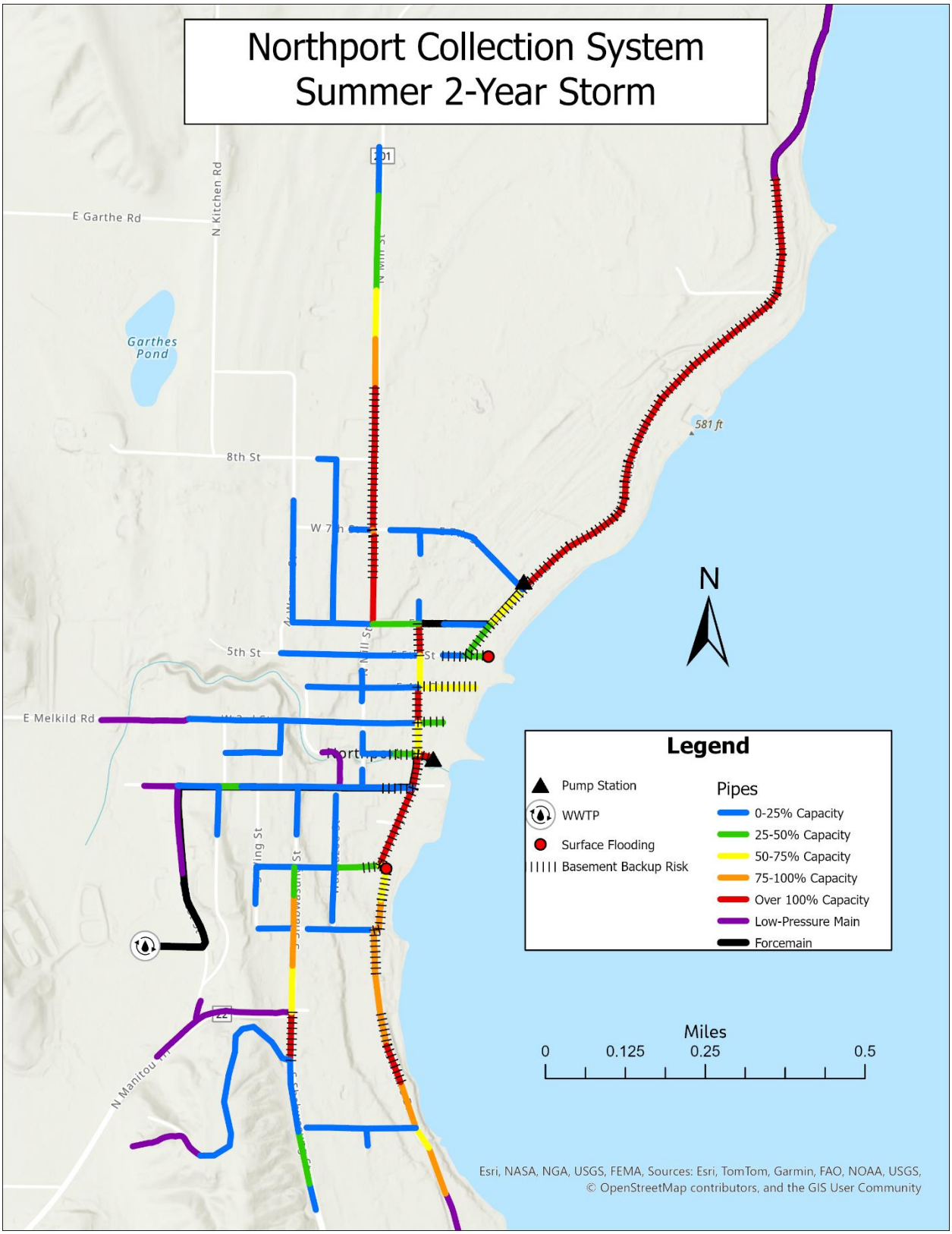

Figure 10-3. Summer 2-Year, 24-Hour Storm Results

10.5 Summer 5-Year, 24-Hour Storm Event Results

Table 10-2 shows the model-predicted maximum inflow and maximum pumping rate of the three pump stations (PS) during the 5-year, 24-hour storm event, along with the capacity of each PS.

Like the 2-year, 24-hour storm, the modeled maximum inflow at PS02 and PS03 exceed the pump station capacity for the 5-year, 24-hour storm event, indicating that the respective pump stations are undersized to convey the flow getting to the pump station for the respective storm event. The modeled maximum inflow at PS01 does not exceed the capacity of the pump station; however, the modeled maximum pumping rate is lower than the maximum total inflow, indicating that the modeled pumps are not operating at the design condition. This results in surcharge upstream of the pump station, which can cause potential basement backups and flooding manholes.

Table 10-2. Summer 5-Year, 24-Hour Storm Pump Station Performance

Pump Station Name	Pump Station Capacity (gpm)	Maximum Total Inflow (gpm)	Maximum Pumping Rate (gpm)
PS01 (Main)	2,700	1,202	885
PS02 (7th Street)	390	750	268
PS03 (NPP)	100	161	120

Similar to the 2-year, 24-hour storm event, the 5-year, 24-hour storm results map, shown in Figure 10-4, reveals capacity issues along North Mill Street, North and South Shore Drive, North Bay Street, and South Shabwasung Street. Houses on these streets are at risk of basement backups. Six manholes along these streets or nearby intersections are predicted to have surface flooding during the 5-year, 24-hour event. Forty-two (42) pipe segments in the system were identified to exceed their calculated flow capacity during the storm.

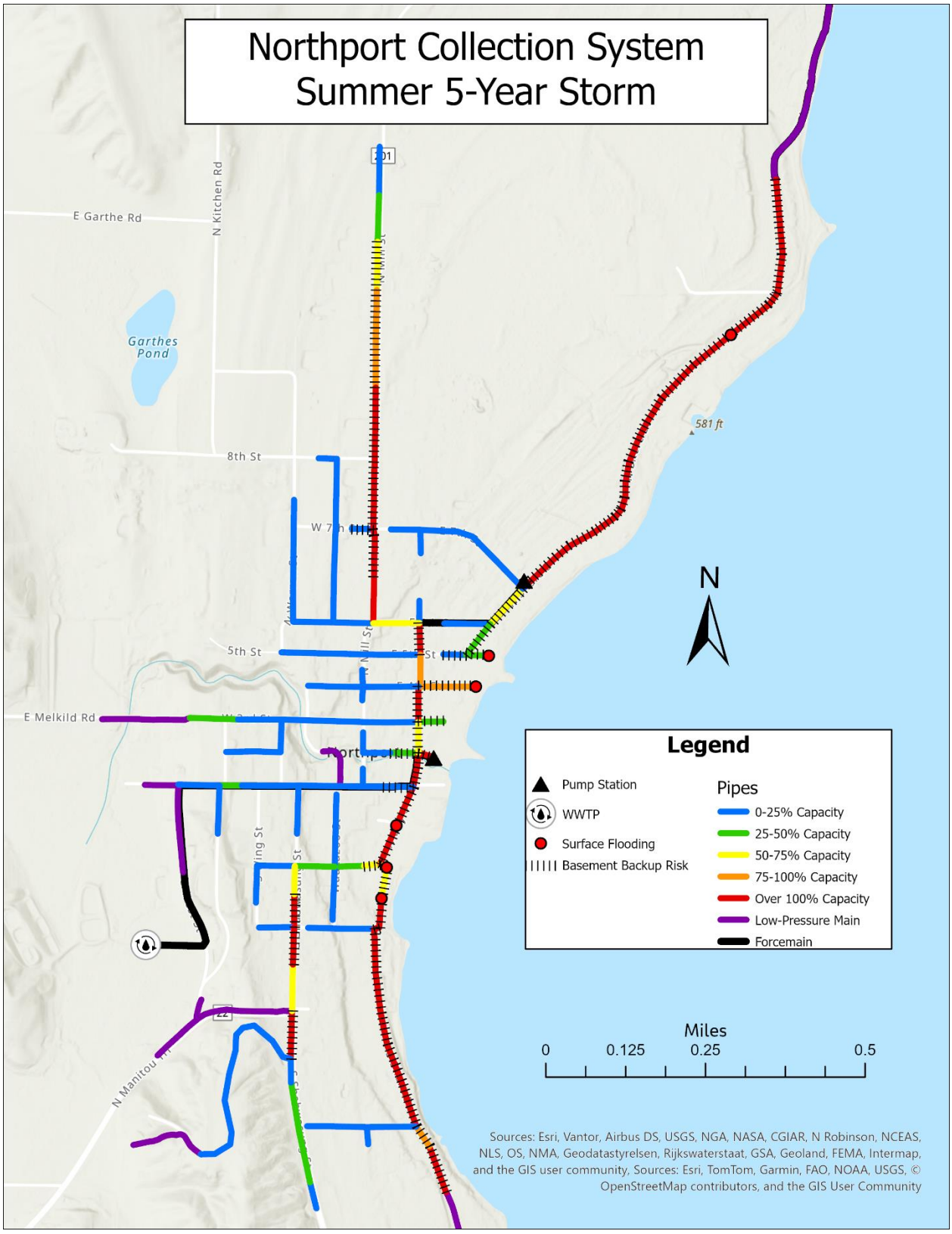

Figure 10-4. Summer 5-Year, 24-Hour Storm Results

Table 10-3 summarizes the results of the collection system capacity analysis. The model results indicate that the dry weather scenarios pose no capacity issues. For the 2-year, 24-hour storm event during both summer and winter conditions, the model indicates that 22% of the gravity pipes in the system by linear feet (LF) exceed capacity and 32% of the gravity system by LF is vulnerable to basement flooding. For the summer 2-year, 24-hour storm event two manholes exhibit surface flooding for a total of approximately 0.03 million gallons (MG). During the winter 2-year, 24-hour storm event one manhole flooded to produce a flood volume of 0.01 MG. For the 5-year, 24-hour storm event during both summer and winter conditions, the model indicates that 29% of the gravity pipes in the system by LF exceed capacity 41% of the gravity system is vulnerable to basement flooding, and six manholes exhibit surface flooding. The projected total flood volume during the 5-year, 24-hour storm event is approximately 0.14 MG for summer and 0.13 MG for winter.

Table 10-3. Collection System Model Results Summary

Scenario	Pipes Over Capacity (% of LF)	Basement Backup Risk (% of LF)	Number of Flooding Manholes[a] (#)	Flooding Manhole Volume[a] (MG)
Summer Dry Weather	0%	0%	0	0
Summer 2-Year Storm Event	22.1%	32.4%	2	0.03
Summer 5-Year Storm Event	29.0%	41.0%	6	0.14
Winter Dry Weather	0%	0%	0	0
Winter 2-Year Storm Event	22.1%	32.4%	1	0.01
Winter 5-Year Storm Event	29.0%	41.0%	6	0.13

[a]Flooding manholes defined using flood volume threshold of 0.01 MG.

11. Collection System Conclusions and Recommendations

The collection system evaluation used the developed model to quantify existing capacity and identify problem areas. The summer (busy-season), defined as June through August, experiences higher wastewater volume which produces slightly more flood volume than winter (off-season). Overall, modeling results indicate the season has minimal impact on the identified capacity issues within the collection system. Dry weather results show no capacity issues in the collection system. The 2-year, 24-hour storm and the 5-year, 24-hour storm results indicate capacity issues such as pipes exceeding their calculated flow capacity, basement backup risk, and surface flooding at manholes. These issues were concentrated primarily along North Mill Street, North and South Shore Drive, North Bay Street, and South Shabwsasung Street. The capacity analysis found that PS02 and PS03 are under-capacity for both the 2-year, 24-hour and 5-year, 24-hour storm events. Additionally, while the flows upstream of PS01 do not exceed pump station capacity for the evaluated events, there is back-up upstream of the pump station likely caused by pump station operations.

The model was developed using the best available data at the time of development and analysis. Where data gaps existed, reasonable engineering assumptions were applied based on professional judgment and standard industry practice. It is important to understand that the results presented in this analysis represent model-predicted conditions, and may not fully reflect actual system performance during a 2-year or 5-year (or more intense) storm event. It is recommended that NLTUA review and evaluate any observed flooding or basement backup data available for recent documented storm events and document and evaluate any observed flooding or basement backup events that may occur in the future. Future capital improvement planning for the wastewater collection system should be guided by documented, site-specific flooding observations to ensure that investments target verified system vulnerabilities.

References

[1] FLEIS & VANDENBRINK ENGINEERING INC. , SRF PROJECT NO. 5254-01, WWTP RECORD DRAWINGS, 2008.

[2] FLEIS & VANDENBRINK ENGINEERING INC., NLTUA WWTF OPERATION & MAINTENANCE MANUAL, 2008.

[3] FLEIS & VANDENBRINK ENGINEERING INC., SRF PROJECT NO. 5254-01, COLLECTION SYSTEM RECORD DRAWINGS, 2008.

[4] National Oceanic and Atmospheric Administration (NOAA), National Centers for Environmental Information. https://www.ncei.noaa.gov/access/past-weather/45.22740161069239,-85.79144394819815,45.06717982509467,-85.37496049969332

[5] National Oceanic and Atmospheric Administration (NOAA), Hydrometeorological Design Studies Center. Precipitation Frequency Data Server (PFDS): Michigan. https://hdsc.nws.noaa.gov/pfds/pfds_map_cont.html?bkmrk=mi [hdsc.nws.noaa.gov]

Appendix A: Groundwater Discharge Permit

Appendix B: Process Flow Diagram

Appendix C: Winter Scenario Collection System Maps

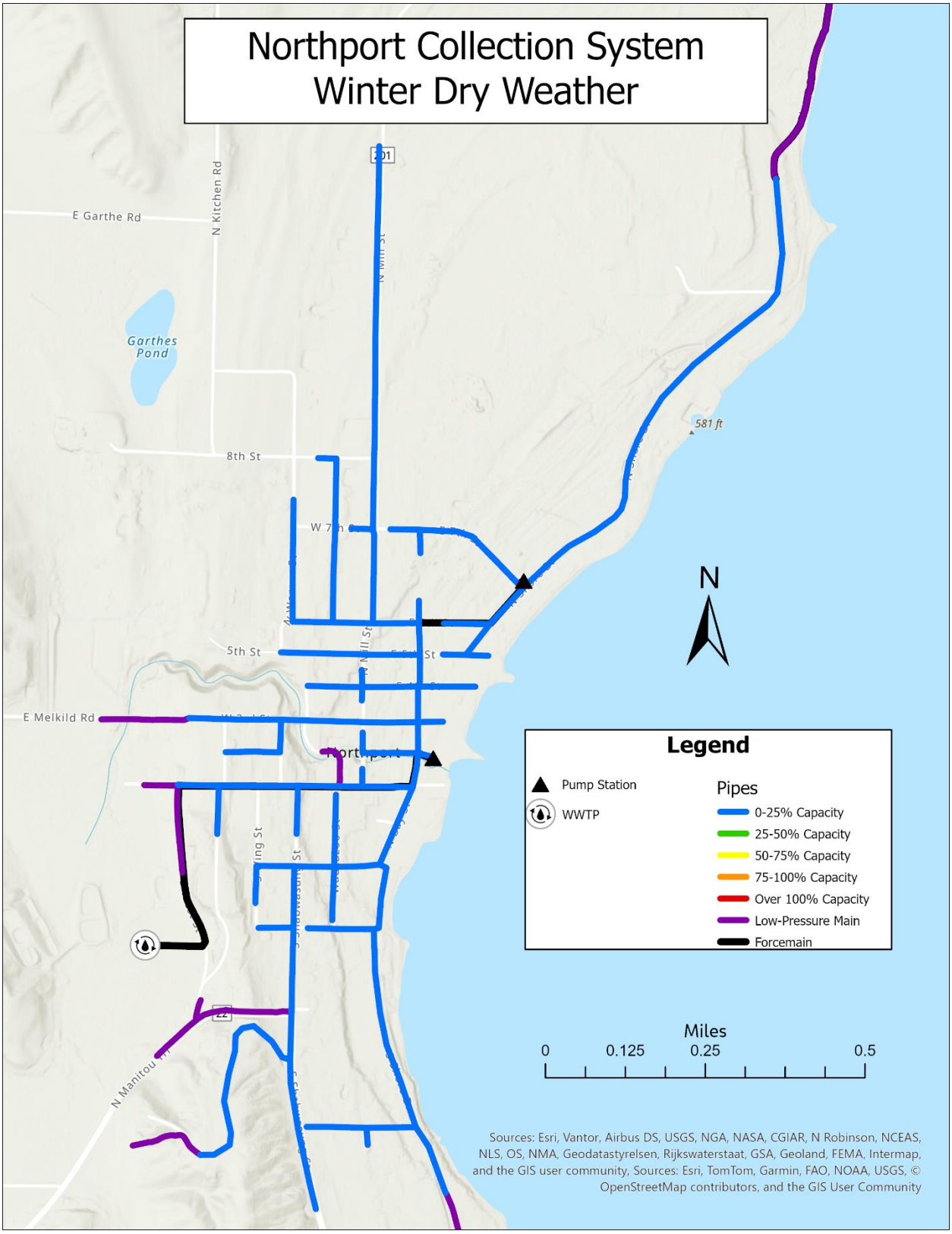

Figure C-1. Map of the Winter Dry Weather Results

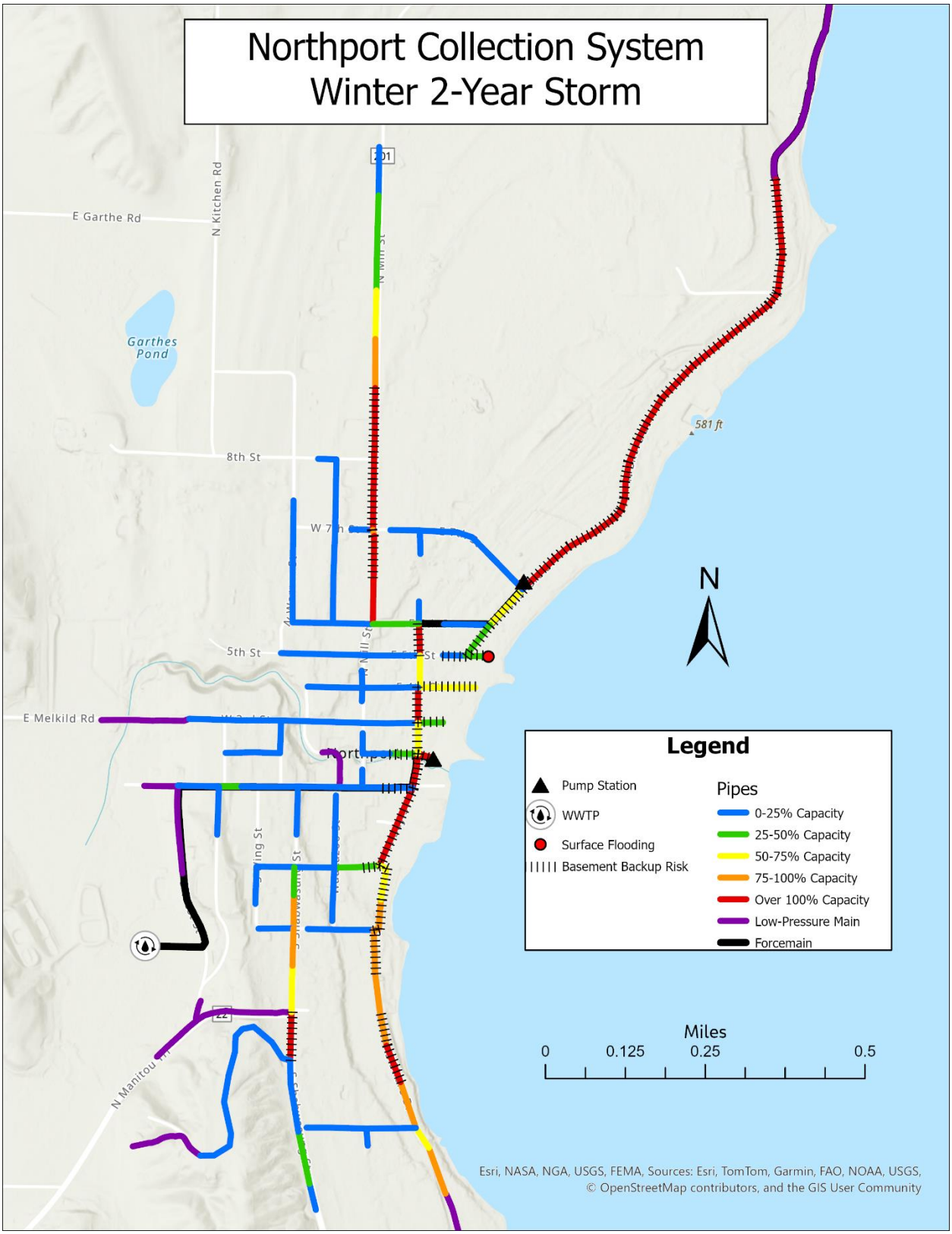

Figure C-2. Map of the Winter 2-Year Storm Results

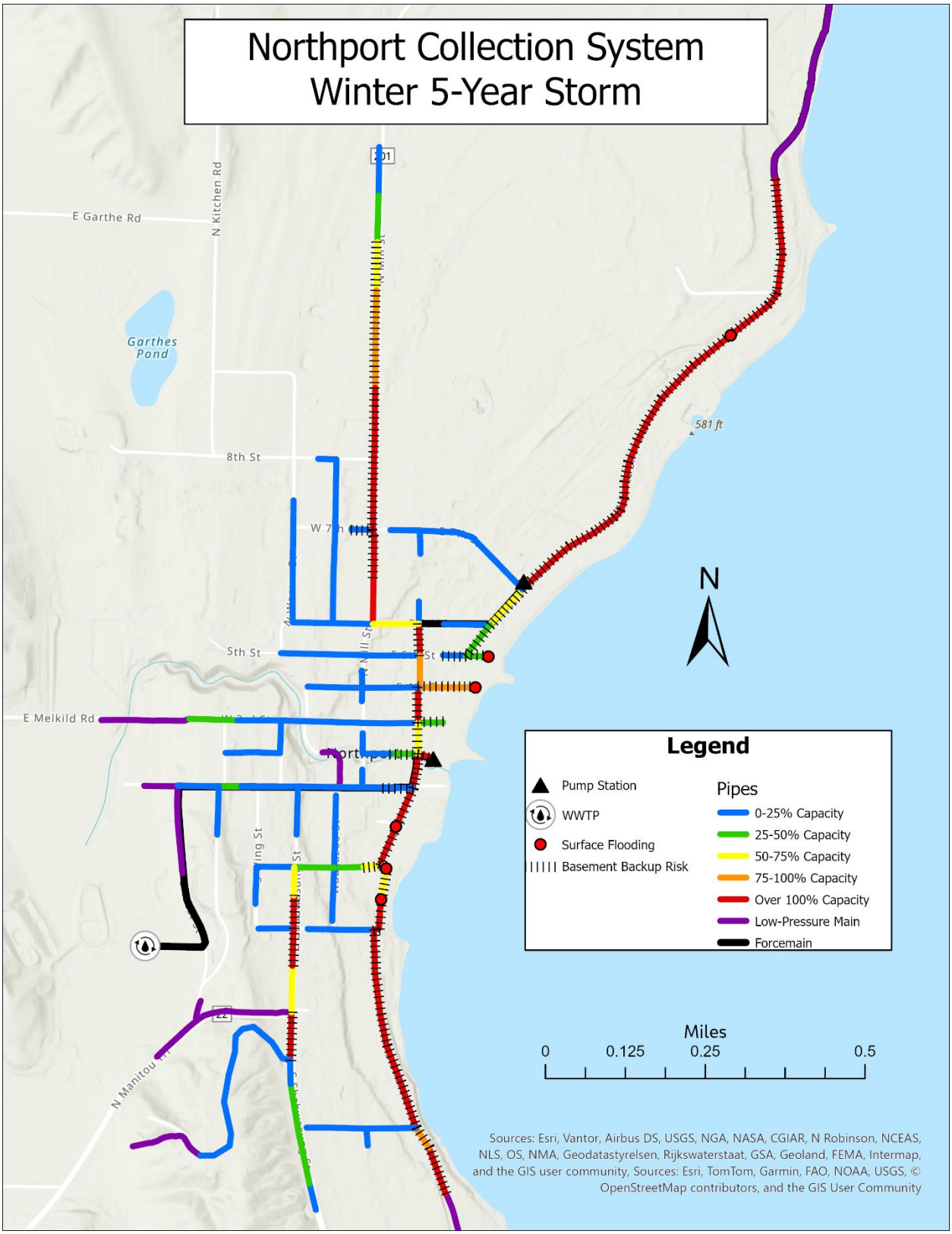

Figure C-3. Map of the Winter 5-Year Storm Results