SANITARY SEWER COLLECTION SYSTEM ASSESSMENT

TOWN OF WATERFORD SANITARY DISTRICT NO. 1
RACINE COUNTY, WISCONSIN
JANUARY/2024

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INTRODUCTION

The Waterford Sanitary District was constructed in the late 1980s to provide sanitary services to the residential areas on both sides of Tichigan Lake and a portion of the Fox River. From the District, sewage is pumped to the Western Racine County Sanitary District for treatment prior to discharge in the Fox River. The system consists of lift stations, gravity sewers, force mains, grinder pump systems, and private property laterals, with several miles of pipe. The system is primarily constructed of PVC with diameters ranging from 8 to 18 inches for the gravity system.

There are 20 lift stations in the collection system. These lift stations are of a dual submersible pump design. Lift station design flow rates range from 100 gallons per minute (GPM) to 700 GPM. Most of the existing lift stations have remained unmodified since the late 1980's. Lift Stations 10 and 15 were renovated in recent years with new wet well piping, valve manhole piping and valves, and a new valve manhole structure. Improvements at other lift stations were maintenance related, consisting of as-needed pump replacements, motor replacements, and pump impeller replacements.

The sanitary sewers have been maintained and improved over the years to increase the service area for new developments or service residences previously on septic. The sanitary sewers currently serve approximately 1,900 homes. Appendix A shows a system map of the Sanitary District with gravity sewers, force mains, grinder pump stations, and sewersheds for each lift station area. This map also shows the locations of the temporary flow meters described. Appendix B shows the pumping scheme of the collection system.

PURPOSE OF STUDY

This study aimed to identify key factors impacting the sanitary sewer and to determine its current capabilities for average and peak hour flow rates, especially regarding future capacities. Several sources of data were used to determine the peak hour flow rate at each individual lift station and the cumulative flow at larger downstream sewers. This allowed for the determination of excess capacity (or deficit) in the system. Additionally, this study aimed to determine if the system can accommodate additional development. With these determinations, this study can serve as a planning document for boundary work for alignment between the Sanitary District and the Western Racine County Sanitary District (WRCSD).

BACKGROUND

To date, there has been no comprehensive capacity analysis conducted on this system. The sanitary sewer system has largely been able to handle strain. However, during storm events, pumping trucks have been needed at select lift stations, especially the larger downstream locations. During the September 11-13, 2022, storm event, roughly 117,600 gallons were pumped from the system at Lift Station 15. The storm that occurred during this time constitutes a 10 to 20-year event and serves as a significant data point in this study. Typically, sanitary sewer capacity studies are based on ten-year storm events. As further development is planned, this study is necessary to confirm the capacity of this system at both present and future conditions.

CAPACITY METHODOLOGY

As a first step for the system capacity determination, the individual flow rates of each lift station were determined. For this, draw-down testing was performed at each lift station. Draw-down testing is a means of determining the actual flow rate through geometry. Each foot of depth in the wet well has a corresponding volume based on its shape and size. First, the inflow rate to the wet well was determined. Over a set period, water was allowed to flow into the system. The height of the water was measured before and after this time. Thus, volume over a period (GPM) could be determined. Next, the same procedure was performed for the outflow. The inflow rate determined earlier was added into the equation to properly account for the total flow rate.

As the system is made up of daisy-chained lift stations, testing runs for downstream stations were timed so that other stations would not be pumping into them. For the farthest downstream lift stations (i.e., 10 and 15), it was not possible to completely avoid upstream flow. However, runs were timed so that only small lift stations would be pumping to minimize potential errors in the data. The larger upstream stations were turned off.

These calculated pumping rates were compared to the design flow rates made available by the Waterford Sanitary District. A summary of the design lift station capacity against the actual lift station capacity is shown in Table 1. Pumps that did not meet the design criteria are marked in red. Differences between the pumping rates could be due to impeller wear and/or partial impeller clogging. Theoretical flow rates are from pump catalog data.

Table 1- Theoretical and Draw Down Lift Station Pump Capacity

Lift Station No.	Theoretical Flow Rate (gpm)	Draw Down Pump 1 Rate (gpm)	Draw Down Pump 2 Rate (gpm)
1	100	106	120
2	100	105	117
3	100	104	66
4	100	268	303
5	100	112	130
6	300	186	203
7	450	470	438
7A	100	43	85
8	100	867	110
9	350	297	382
10	500	584	545
10A	100	167	91
10B	100	83	68
11	100	112	109
12	150	123	117
13	100	168	180
14	100	152	184
14A	200	148	146
15	700	670	666
15A	100	111	138

FORCE MAIN CAPACITY

The force mains that comprise the sewer system directly downstream from the lift stations were also studied. Each size of pipe has a known maximum recommended flow. At a maximum flow equivalent to 6 feet per second velocity, water hammering is a non-issue. Water hammer is the phenomenon of separation of the water column inside the force main due to a sudden change in the pumping rate. Water hammer is of most concern when a pump suddenly turns off due to power loss. This can be the case during severe weather (e.g., thunderstorms) when the lift stations are experiencing elevated flow rates from inflow. Limiting the maximum flow rate and velocity thus protects the joints and the system in case of a power outage. Recommended minimum and maximum flow rates after each lift station are shown in Table 2.

Table 2-Lift Station Force Main Size and Recommended Flow Rates

Lift Station No.	Force Main Size	Minimum Flow (GPM)	Maximum Flow (GPM)
1	4"	80	240
2	4"	80	240
3	4"	80	240
4	4"	80	240
5	4"	80	240
6	6"	180	530
7	8"	310	950
7A	4"	80	240
8	4"	80	240
9	8"	310	950
10	10"	500	1450
10A	4"	80	240
10B	4"	80	240
11	4"	80	240
12	4"	80	240
13	4"	80	240
14	4"	80	240
14A	6"	180	530
15	12"	700	2100
15A	4"	80	240

GRAVITY SEWER CAPACITY

To calculate the theoretical capacity of the gravity sewer lines, Manning's equation was used. Manning's equation is a formula that can determine the flow rate using the pipe material, the size, and slope of the pipe. The pipe material used was PVC, along with an N coefficient of 0.011. Using the Waterford Sanitary District's ArcGIS system, the size and slope of the gravity sewers could be determined. Values were confirmed using record drawings and values from an R/M field survey, where measure-down readings were taken at select manholes. The formula was applied to pipes greater than or equal to 12-inch diameter, especially those directly upstream of the lift stations. These were primarily in the southeastern part of the District. Calculated minimum and average capacities are shown in Table 3. The average capacities in the table are comprised in general, of three manholes spans in the area. This was also the basis for the minimum value.

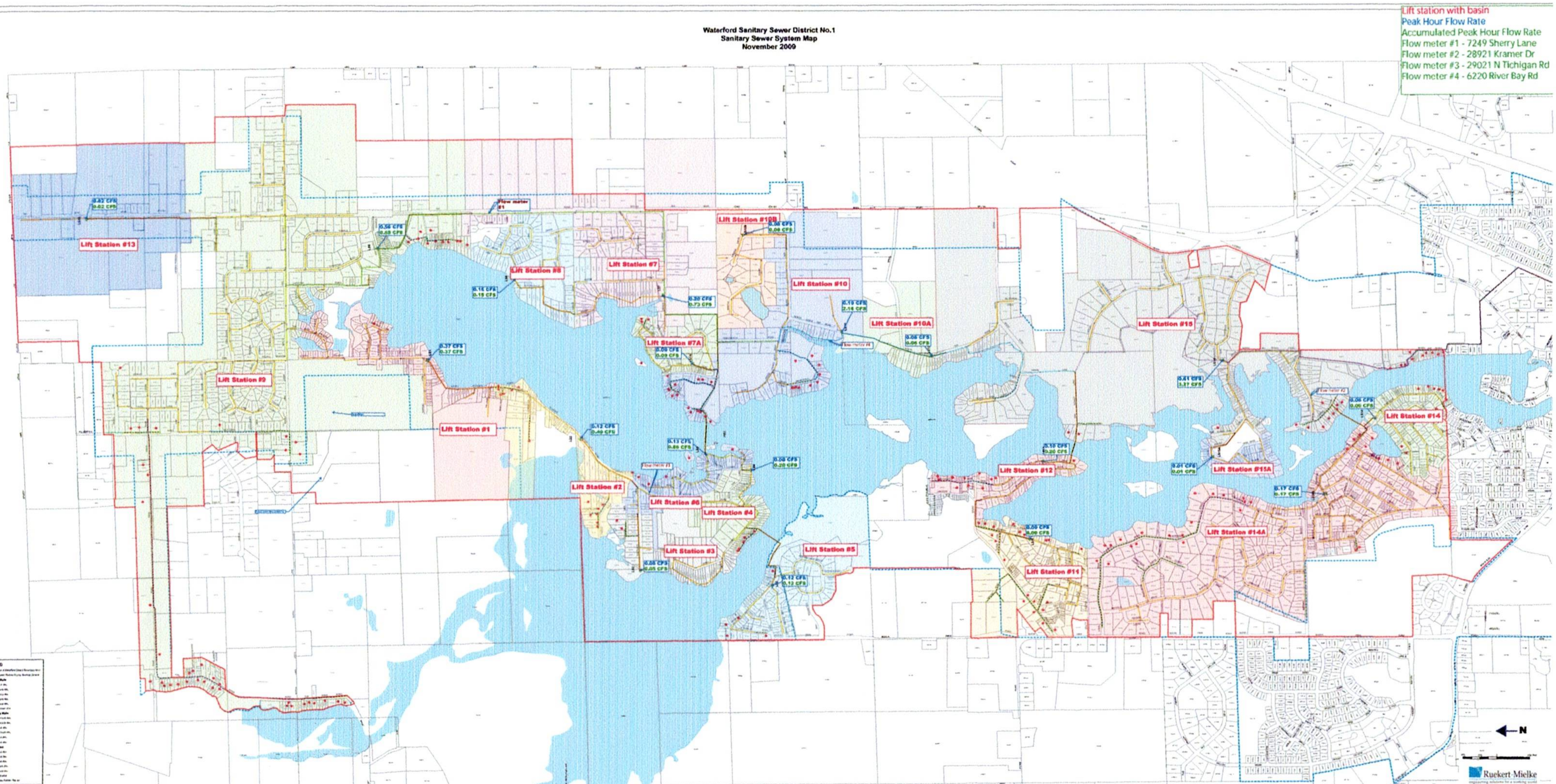
Table 3- Calculated Minimum and Average Capacities of Gravity Sewer Lines

Gravity Sewer Road Location	Location Relative to Lift Station	Minimum Capacity (GPM)	Average Capacity (GPM)
Lake View Rd	Upstream to 9	909	1018
Cedar Ln	Upstream to 9	752	847
Big Bend Rd	Downstream of 9	997	1137
Beach Dr	Upstream of 7	999	1930
Washington Rd	Downstream of 7	2392	2565
N River Bay Rd	Upstream of 10	1160	2182
Wood Dr	Downstream of 10	1317	1384
Indian Ln	Downstream of 10	1404	2084
Wood Lily Ln	Upstream of 15	1957	2194
Frasier Trail	Downstream of 15	2134	2224

APPENDIX D

LIFT STATION PEAK FUTURE CONDITIONS

Waterford Sanitary Sewer District No.1
Sanitary Sewer System Map
November 2009



FLOW METERING METHODOLOGY

Flow meters were installed throughout the sewer system from April 1st to July 15th, 2023. These meters provided critical information for the determination of the system's capacity. The flow meters used in this study were of the area-velocity type. In meters of this type, a submersible probe measures fluid depth and average velocity. Given the pipe size, a cross-sectional area occupied by fluid can be determined using measured depth. This area multiplied by the measured velocity gives a fluid volume passing through the pipe. This instantaneous flow data was logged, and the data was downloaded once a month. Flow Meter #1 read high on average due to standing water in the system. A corresponding correction factor of 15% was applied to the measured values for this meter.

It was not feasible to install flow meters at every lift station. Four key locations were chosen to install flow meters at. Flow Meter 1 was installed downstream of Lift Stations 9 and 13 on Big Bend Road. Flow Meter 2 was installed downstream of Lift Stations 14 and 14A on Kramer Drive. Flow Meter 3 was installed downstream of Lift Stations 1, 2, and 3 on North Tichigan Road. Flow Meter 4 was installed directly upstream of Lift Station 10 near North River Bay Road. These locations were chosen to maximize the upstream lift stations and to provide tiered data collection. For example, Flow Meter 4, which measures the influent into Lift Station 15, was downstream of Flow Meters 1 and 3. Values were thus able to be confirmed at several locations throughout the system. This allowed for the extrapolation of flows at each lift station.

Over the period these flow meters were installed (April-July 2023), data was collected for average day, peak day, and peak hour conditions. An instantaneous flow reading was taken every five minutes. Data behaved as expected, with significantly higher values in the April rainy season and with higher groundwater levels in the spring. Peak hour events were associated with rainfall events. Table 4 shows the Average Day, Peak Day, and Peak Hour flow rates at each location.

Table 4- Average Day, Peak Day, and Peak Hour Flow Rates from Flow Meters (GPM)

Average Day	**April**	**May**	**June**	**July**	**Average**
Flow Meter 1	74	62	55	61	63
Flow Meter 2	41	39	40	40	40
Flow Meter 3	38	27	30	35	32
Flow Meter 4	272	255	238	241	251
Peak Day	**April**	**May**	**June**	**July**	**Average**
Flow Meter 1	131	78	67	77	88
Flow Meter 2	62	49	48	45	51
Flow Meter 3	83	33	38	43	49
Flow Meter 4	458	297	271	276	326
Peak Hour	**April**	**May**	**June**	**July**	**Average**
Flow Meter 1	188	138	129	140	149
Flow Meter 2	98	86	84	79	87
Flow Meter 3	119	59	90	73	85
Flow Meter 4	545	382	345	341	403

This flow meter data was used to determine average flow rates on a per capita basis. This allowed for accurate modeling of growth and added development. Each dwelling in a basin's area was counted, and the average population factor of 2.8 people per dwelling was used to determine the basin population. The average population factor was taken from the US Census for the Waterford area. The initial average assumption of 100 gallons per capita per day (GPCD) was adjusted to better match flow meter average flow rates at each location. For example, GPCD values for Lift Stations 14 and 14A were heavily adjusted to 65 and 55 GPCD, respectively. Flow due to inflow and infiltration makes up a large proportion of the typical 100 GPCD assumption. As the sewersheds surrounding those lift stations have a high density of low-pressure sewer systems with minimal pipe joints, inflow and infiltration is minimized. The GPCD values were decreased to match the average flow meter values. Table 5 contains the flow factor values used in this analysis.

Table 5- Lift Station Per Capita Flow Factor Values

Lift Station No.	Per Capita Flow Factor (GPCD)
1	80
2	80
3	80
4	115
5	115
6	115
7	115
7A	115
8	115
9	90
10	115
10A	115
10B	115
11	115
12	115
13	90
14	65
14A	55
15	115
15A	115

FLOW RATES AT EACH LIFT STATION

Current average flow rates at each lift station were calculated using the GPCD values and population values. Given the lift station schematic diagram in Appendix B, average flow rates from each basin were added to determine cumulative flow rates.

SCADA pump runtime records from each of the lift stations were also available from the 10–20-year event in September 2022 and a typical day on 6/22/2022. It was assumed that 6/22/2022 represented an average day flow rate condition. These SCADA pump runtime records, along with the average pump output from the drawdown tests, gave peak and average daily flow rates for each lift station. These values were used as a check against the calculated flow rates.

To determine peak hour flow rates at each station, the ratio of peak day: average day pump run-times was calculated for each station. A 1.5 ratio was assumed and used for peak hour: peak day conditions. This value was generally supported by the measured flow meter data. However, some lift station tributary areas had a lower ratio. To be conservative, the 1.5 factor was kept.

Using the peak hour factor, and the average flow rates, the peak hour flow rates for each basin were determined. In the case of Lift Station 15, an additional factor of 117,600 gallons was added based on the data from 9/11/2022. The same population-based approach was employed to determine current peak hour cumulative flow rates. To be conservative, attenuation was not considered. Lift Station 15 was the furthest downstream station, and consequently had the highest flow. Table 6 contains the average flow rate, peak hour flow rate, and peak hour: average day ratio for each lift station at existing conditions.