Water System Master Plan Report

Village of Tinley Park, Illinois

~~May~~ 2021

BAXTER & WOODMAN
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Village of Tinley Park
Water System Master Plan Report
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LIST OF ABBREVIATIONS

AWWA	-	American Water Works Association
avg	-	average
CIP	-	Capital Improvement Plan
CPM	-	cycles per minute
DI	-	ductile iron
DWC	-	DuPage Water Commission
ft.	-	feet
gcd	-	gallons per capita per day
GIS	-	geospatial information system
gpd	-	gallons per day
gpm	-	gallons per minute
HMI	-	Human Machine Interface
HSP	-	high service booster pumping
IDNR	-	Illinois Department of Natural Resources
IEPA	-	Illinois Environmental Protection Agency
ISO	-	Insurance Service Office
KPI	-	Key Performance Indicators
kW	-	kilowatt
lbs	-	pounds
MCC	-	motor control center
MG	-	million gallons (or mil gal)
MGD	-	million gallons per day
mg/L	-	milligrams per liter (parts per million in dilute solutions)
MUL	-	Maximum Unavoidable Leakage
PLC	-	Programmable Logic Controller
PS	-	pumping station
psi	-	pounds per square inch
PVC	-	polyvinyl chloride
RSWW	-	Recommended Standards for Water Works
SCADA	-	supervisory control and data acquisition
UFF	-	Unaccounted-For-Flow
VHF	-	very high frequency
VFD	-	variable frequency drive
WRF	-	water reclamation facility

EXECUTIVE SUMMARY

The Village of Tinley Park's potable water supply system consists of one elevated storage tank, four ground level reservoirs, two pumping stations, and approximately 270 miles of water main. The Village receives Lake Michigan water through two primary interconnections with the Village of Oak Lawn. The water system currently provides 8.3 MGD of water on an average day; 4.10 MGD is used by Tinley Park's approximately 56,000 residents and an additional 3.78 MGD passes through to the Villages of New Lenox and Mokena, and about 0.43 MGD passes through to the Village of Orland Hills. During the summer months, maximum daily water pumpage can increase to over 15.3 MGD. A map of the water system including major facilities was developed and included as Exhibit A. A schematic of the Village's Water System is included in Appendix A.

The Village's objective for the Water System Master Plan is to complete a thorough review of the entire water system facilities and operations to achieve a comprehensive water system planning document for water operations, facilities, and the distribution system. The purpose of this report is to develop a systematic approach for making both short and long term capital and operational efficiency improvements to help the Village build a roadmap for future improvements.

The Water System Master Plan included the following components:

1. Review of past water use and future water demand projections.

2. Review of distribution system data and development of a hydraulic distribution model developed from the Village's Geographic Information System (GIS) database.

3. Development of infrastructure needs including estimated project costs.

4. Analysis of pressure characteristics, excessive head loss or high velocities, areas with inadequate fire flows , and existing critical facilities within the water model.

5. Utilization of the water model and the Village's historical main break data to prepare a water main break analysis and recommend a priority ranking for water main replacement.

6. Extended period simulation analysis to evaluate the Village's distribution system in a real time mode for a 96 hour period in order to identify problems areas in the existing system during a variety of flow conditions, hourly demands, and control settings.

7. Review of the total volume of water storage available in the distribution system and compare to existing and ultimate maximum day and peak hourly water demands.

An updated WaterGEMS® hydraulic model of the Village's water distribution system was developed and utilized to assist with verification of system pressures, fire flow capabilities, and development of water main improvements along with review of alternative operational schemes. The new water model correlates directly with the Village's GIS. Data from the model can be linked directly to GIS so that pressures and fire flows are readily available to Village staff.

The water system is mostly capable of meeting required system pressures set forth by the Illinois Environmental Protection Agency (IEPA) which range from a minimum of 35 psi to a maximum of 100 psi. However, areas in the far west of the system have difficulty maintaining minimum pressures of 35 psi during maximum daily demands. Solutions to these low pressures include creating a second pressure zone equipped with a booster station in conjunction with three pressure reducing valve (PRV) stations.

Recommended distribution system improvements were developed and simulated with the model to address system deficiencies. Maps of the individual water main improvements and a summary of recommendations with total project costs are provided. Near term projects were aligned with proposed street project plans. Long-term future projects were listed in order of priority to provide insight during the project planning process and also allow for flexibility in future improvement programs.

Select major distribution improvements are noted in Table 1 along with budgeted costs to cover expected cost of construction, bidding, and engineering.

TABLE 1

Recommended Improvements (2021-2025)

Budget Year	Proposed Improvements	Proposed Diameter (in)	Length (ft)	Estimated Total Project Cost
2021	La Grange Rd Utilities Extension	12	2,700	$1,060,000
	Proposed West Pressure Zone	-	-	$2,070,000
	S La Grange Rd & 175th St	12	2,000	$860,000
	La Grange Road Gap	12	600	$260,000
	2021 SUBTOTAL			**$4,250,000**
2022	67th Court - 175th St to 174th St	12	1,100	$470,000
	66th Ct & 173rd Pl	8	700	$260,000
	Dorothy Ln	8	990	$450,000
	2022 SUBTOTAL			**$1,180,000**
2023	179th St & Oak Park Ave	8	1,000	$380,000
	174th Pl - Oak Park Ave & 66th Ave	8	1,100	$410,000
	Ironwood Drive	8	1,400	$530,000
	2023 SUBTOTAL			**$1,320,000**

Budget Year	Proposed Improvements	Proposed Diameter (in)	Length (ft)	Estimated Total Project Cost
2024	**Terrace Dr & Skyline Dr**	**8**	**1,400**	**$530,000**
	160th Place	8	1,200	$450,000
	Bremen Grove Loop	12	900	$390,000
			2024 SUBTOTAL	**$1,370,000**
2025	176th Street	8	1,300	$480,000
	Ozark Ave & 159th Pl	8	1,200	$450,000
	176th Place	8	900	$330,000
			2025 SUBTOTAL	**$1,260,000**
	2021-2025 CAPITAL IMPROVEMENTS PLAN TOTAL			**$9,380,000**

1. INTRODUCTION

1.1 Background

The Village of Tinley Park is committed to continuing the high quality of life enjoyed by the citizens and businesses of the Village. Completing the Village's Water System Master Plan is a step towards fulfilling that mission by ensuring that its citizens will have an ample supply of high quality water at a reasonable cost. A detailed water system plan is needed to support fiscally sound and responsive decisions while focusing on capital and operational efficiencies.

The Village of Tinley Park owns and operates a well maintained water distribution, pumping, and storage system. The Village's objective is for the Master Plan project to be a thorough study, review, and analysis of the entire water system to achieve an efficient, economical water system plan for operations, facilities, and the distribution system.

1.2 Study Purpose and Goals

The purpose of the Village's Water System Master Plan Report is to develop a systematic approach for making capital improvements and operational modifications intended to meet the Village's water supply needs and optimize operations. The Village's primary concerns include meeting or exceeding water quality standards; ensuring reliable supply and service to its residents, businesses, and potential customers; maintaining adequate pressures and fire flows; evaluating system operational changes; and improving the system efficiency and cost effectiveness.

The Master Plan Report will be a valuable tool for making short term and long term operational improvements. This Master Plan Report includes the following:

1. Estimates of existing residential population, non-residential population equivalents, and water demands.
2. Updated projected population estimates based on best available data.
3. An updated WaterGEMS® hydraulic water model based on the Village's GIS and field fire hydrant flow testing.
4. Use of the newly created hydraulic water model to identify problem areas and proposed solutions within the distribution system under a variety of demand conditions.
5. Recommendations for capital improvements based on results of water system modeling. Estimates of capital costs and prioritization of the distribution system improvements are included.

2. WATER USE AND POPULATION

This section of the report provides information on past water use, population projections, and estimates for future water demands. Past water demand information is based on water pumpage data provided by the Village. The Village's population data (as taken from U.S. Census data) is used to estimate average per capita demands. Future population is derived from the U.S. Census Bureau Forecast of Population and the Village's recent observed growth rate. Estimates of future population are intended for informational use only and have not been used to project specific dates for proposed improvements.

2.1 Water Supply

The Village receives treated Lake Michigan water from the Village of Oak Lawn through two interconnections. These interconnections feed directly to four ground storage tanks split between two pumping stations and are described in Table 2 below.

TABLE 2

Primary Water Supply Facilities

Description	Location	Size (in)
Post 1 Oak Lawn Feed	6612 167th St	30
Post 2 Oak Lawn Feed	18308 S Ridgeland Ave	36

2.2 Water Pumpage Data

Table 3 shows the Village's average and maximum day (the highest total pumpage over a 24-hour period) demands for the last five years, along with a calculated ratio of the maximum to average day peaking factor. Plots and analyses of Tinley Park's water pumpage data are presented in more detail in subsection 2.4.

TABLE 3

Yearly Water Pumpage Data

Year	Average Day Pumpage (MGD)	Maximum Day Pumpage (MGD)	Maximum Day Peaking Factor
2015	8.43	16.1	1.91
2016	8.53	17.0	2.00
2017	8.31	13.7	1.65
2018	8.29	15.0	1.81
2019	8.02	14.5	1.81
Average	**8.31**	**15.3**	**1.83**

Between 2015 and 2016, the maximum day demand remained consistently above 16 MGD. Between 2017 and 2019, the maximum day demand decreased to 14-15 MGD. The average maximum day demand value 15.3 MGD between 2015 and 2019 was utilized for this report. The average maximum day peaking factor is 1.83 which is in the typical range for a community like Tinley Park.

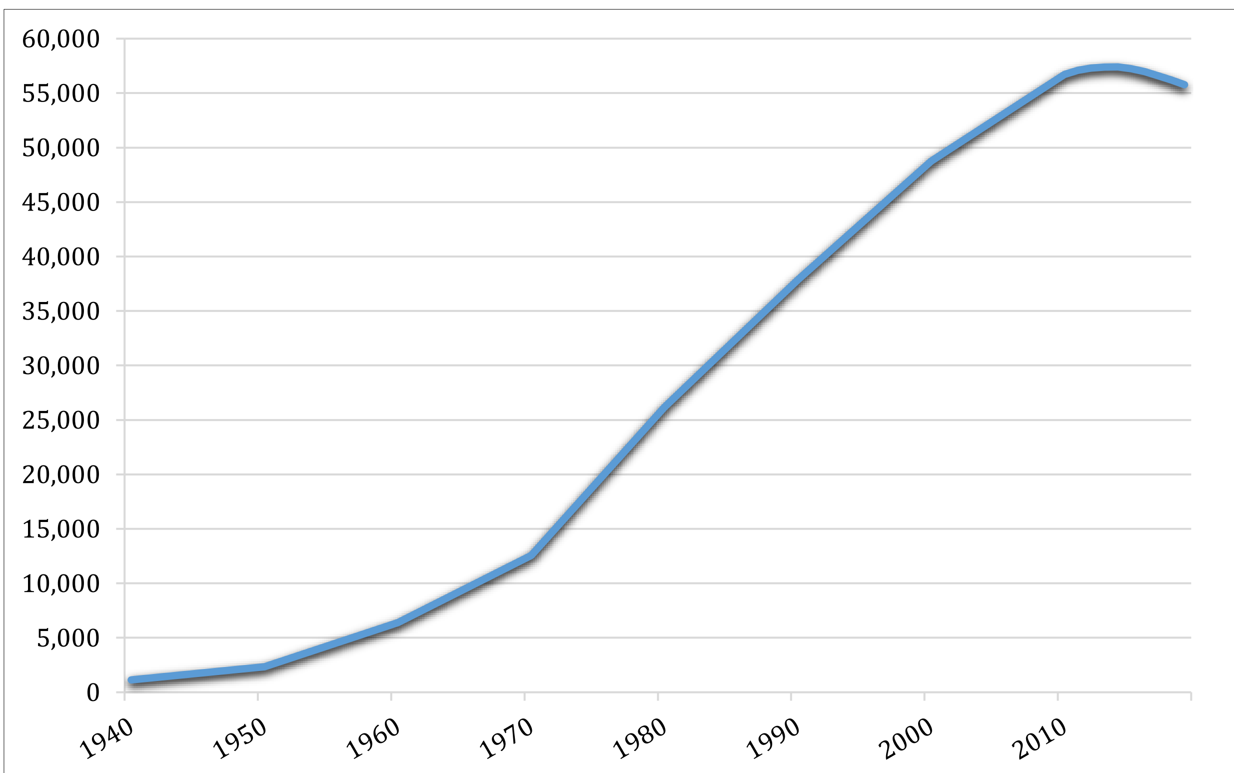
2.3 Population Projections

Water consumption in communities is strongly correlated to population. When reliable population forecasts are combined with past per capita water consumption data, it is possible to reasonably plan for future water demands and recommended facilities to meet those needs. While population projections are never 100% accurate, they do provide for a sound basis on which to estimate and quantify future water supply needs and trends.

Tinley Park's population grew dramatically from the 1960's to 2010, but has tapered off in the time since 2010 to 2020 as shown in Figure 1. The 2010 U.S. Census data estimated the population of Tinley Park at 56,703 people. The population of Tinley Park in 2019 is noted at 55,773 people.

FIGURE 1

Population Trend



The U.S. Census Bureau is currently predicting a 0.3% annual rate of growth for Illinois. However, the Village has experienced a leveling out of population growth in the last 10 years. This report will focus on targeted areas of future development.

In discussions with the Village, known areas of future development are outlined in Table 4 including development size in acres and estimated Population Equivalents (PE) projections using a weighted average of projected land use. Assumptions made for PE per acre estimates for different land uses are outlined in Table 5.

TABLE 4

Future Development Areas

Future Development Area	Area [Acres]	Estimated Population Equivalents
White Eagle Drive	18.5	254.38
S La Grange Rd & 179th St	34.6	519.00
S La Grange Rd & 179th St	20.3	304.50
Near Great Escape on S La Grange Rd	2.3	34.50
Harlem Ave & Vollmer Rd	223.2	2879.28
South St & 67th Court	4.7	62.98
State Mental Health Property	283	4245
TOTAL	**586.6**	**8299.64**

TABLE 5

Population Equivalents Projections by Land Use Category

Land Use Category	PE/Acre
Rural Residential	0.87
Low Density Residential	6
Typical Residential	8
Medium Density Residential	13
Commercial	15
Mixed Use	12.5
Industrial	15
Public	10

Most of the future developments described in Table 4 consist of commercial development and are not associated with population increase but are associated with an increase in water demands. Water demand projections as a result of these future developments are discussed in Section 2.4.

2.4 Water Consumption Data

The water system's average day demands are calculated by dividing the total yearly water demand divided by the number of days in the year. Average per capita water demand data is calculated by dividing the average day demand by the estimated population for the year.

Table 6 shows past average day demand along with average per capita demand data for the past five years. A plot of average day pumpage through year 2019 is shown in Figure 2. A plot of average per capita demand is shown in Figure 3. For purposes of this report, we will assume an average per capita demand at 72.4 gallons per day, based on average use over the last five years.

Average day per capita demand can be calculated by dividing the average day demand by estimated population served. Table 6 shows the average consumption per capita from 2015 through 2019. As shown in Table 6 average consumption peaked in 2016 with a value of 75.2 gpcpd, and has decreased steadily since then. In 2019, the average day per capita demand was calculated as 70.7 gpcpd.

TABLE 6

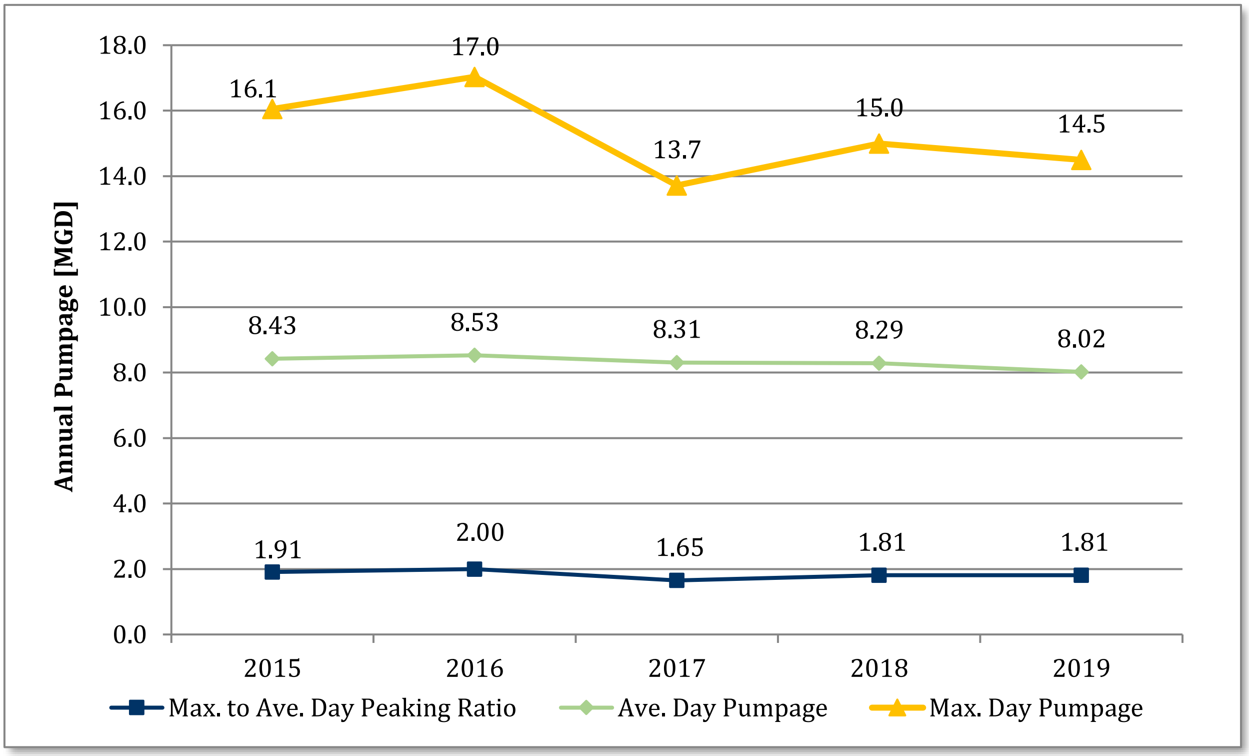
Population, Per Capita, and Average Day Demand Data

Year	Population	Average Per Capita Demand (gcd)	Average Day Demand (MGD)
2015	57,249	74.3	4.25
2016	56,985	75.2	4.29
2017	56,606	72.3	4.09
2018	56,204	69.6	3.91
2019	55,773	70.7	3.94
	AVERAGE	**72.4**	**4.10**

There is no AWWA standard for average water consumption as water use varies from community to community. However, average residential consumption has decreased in the Chicagoland region over the past 20 years from approximately 100 gallons per day to 80 gallons per day. This reduction in water demand can be attributed to factors such as increased education on water conservation and low flow appliances.

FIGURE 2

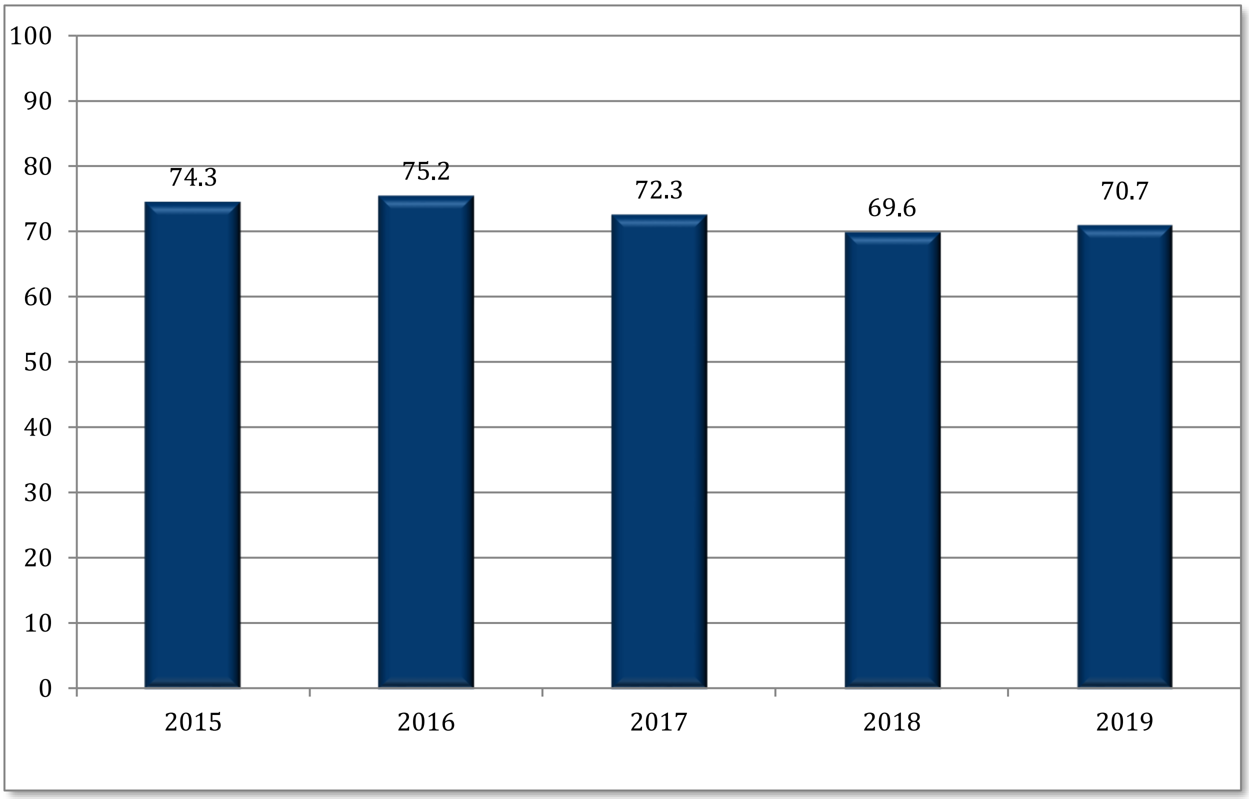
Average Day Pumpage (including wholesale customers)



In addition to a plot of average day pumpage, Figure 2 also shows the plot of maximum day pumpage through year 2019 along with a plot of the ratio of "Maximum Day divided by Average Day" pumpage data. As noted, average day demand has steadily decreased since 2015. The Maximum Day/Average Day Ratio ranged from a low of 1.65 to a high of 2.00, and is similar to other nearby communities. Many Lake Michigan water communities have a 1.65 to 1.80 maximum day/average day contractual limit placed on their maximum daily withdrawals in an effort to limit outdoor water use. For the purpose of this report, and based on the greatest and most reliable Maximum Day Demand for the above time frame, we will assume a 1.83 peaking factor for Tinley Park.

FIGURE 3

Average Day Per Capita Water Demand



IEPA requires that water systems be designed to meet maximum day demands as high demands can occur for many days or weeks in succession. Table 7 displays past maximum to average day peaking ratios and maximum day pumpage for the past five years.

TABLE 7

Population and Maximum Day Pumpage

Year	Population	Maximum to Average Day Peaking Ratio	Maximum Day Pumpage [MGD]
2015	57,249	1.91	16.1
2016	56,985	2.00	17.0
2017	56,606	1.65	13.7
2018	56,204	1.81	15.0
2019	55,773	1.81	14.5

Demand projections for the future development areas described in Section 2.3 are shown in Table 8. Using the Village's per capital water demand of 72.4 gpcd, the additional water demand from these future developments is projected to be approximately 400 gpm on average. This equates to an 0.60 MGD on an average day, 1.11 MGD on a max day, and 1.83 MGD on a peak hour basis.

TABLE 8

Projected Water Demand of Future Development Areas

Future Development Area	Area [Acres]	Estimated PE	Per Capita Demand [gpcd]	Projected Demand [gpm]
White Eagle Drive	18.5	254.38	72.4	12.79
S La Grange Rd & 179th St	34.6	519.00		26.10
S La Grange Rd & 179th St	20.3	304.50		15.31
Near Great Escape on S La Grange Rd	2.3	34.50		1.74
Harlem Ave & Vollmer Rd	223.2	2879.28		144.81
South St & 67th Court	4.7	62.98		3.17
State Mental Health Property	283	4245		213.50
TOTAL				**417.43**
Average Day Demand [MGD]				**0.60**
Max Day Demand [MGD]				**1.11**
Peak Hour Demand [MGD]				**1.83**

A summary of existing and projected future water pumpage is summarized in Table 9.

TABLE 9

Existing and Future Projected Water Demand

Year	Average Day Pumpage (MGD)	Maximum Day Pumpage (MGD)
2015	8.4	16.1
2016	8.5	17.0
2017	8.3	13.7
2018	8.2	15.0
2019	8.0	14.5
Future	8.9	16.4

3. EXISTING WATER SUPPLY SYSTEM

3.1 General Description

The Village of Tinley Park's potable water supply system consists of one elevated storage tank, four ground level reservoirs, two pumping stations, and approximately 270 miles of water main. The Village's Lake Michigan water supply is provided through two primary interconnections with the Village of Oak Lawn. The water system currently provides approximately 8.31 MGD of water on an average day to nearly 56,000 Tinley Park residents and the Village of New Lenox, the Village of Mokena, and the Village of Orland Hills. During the summer months, this demand can increase to over 15.3 MGD. A map of the water system and major facilities is displayed in Exhibit A. The water system is illustrated as a schematic shown in Appendix A. A water distribution facility summary is shown in Table 10.

TABLE 10

Water Distribution Facility Summary

Site Type	Facility Name
Elevated Tanks	Post 11
Above-ground / Ground Level	Post 1 [West] Reservoir
	Post 1 [East] Reservoir
	Post 2 [North] Reservoir
	Post 2 [South] Reservoir
Pumping Stations	Post 1 Pumping Station
	Post 2 Pumping Station
Primary Water Supply Connections	Post 1 Oak Lawn Feed
	Post 2 Oak Lawn Feed

3.2 High Service Pumping Capabilities

High service booster pumping stations (HSP) are located at Post 1 and Post 2. A breakdown of the Village's high service booster pumping capabilities is noted in Table 11 for each of the two total pumping stations.

TABLE 11

Pump Capacity by Location

Location	Pump Capacities (gpm)	Firm Capacities (gpm)
Post 1 Pumping Station	4 pumps @ 4,000	3 pumps @ 4,000
Subtotal	16,000	12,000
Post 2 Pumping Station	4 pumps @ 4,000	3 pumps @ 4,000
Subtotal	16,000	12,000
TOTAL	**32,000**	**24,000**

The "Firm" pumping capacity is calculated when the largest pump is assumed to be out of service. The Illinois EPA requires that a system be evaluated assuming the largest pump is out of service at each pumping station. The Village's total firm pumping capacity is 24,000 gpm (34.6 MGD). The total firm pumping capacity is more than sufficient to meet the total maximum daily demand of 15.3 MGD.

FIGURE 4

Total Pumping Capacity at each Pumping Station

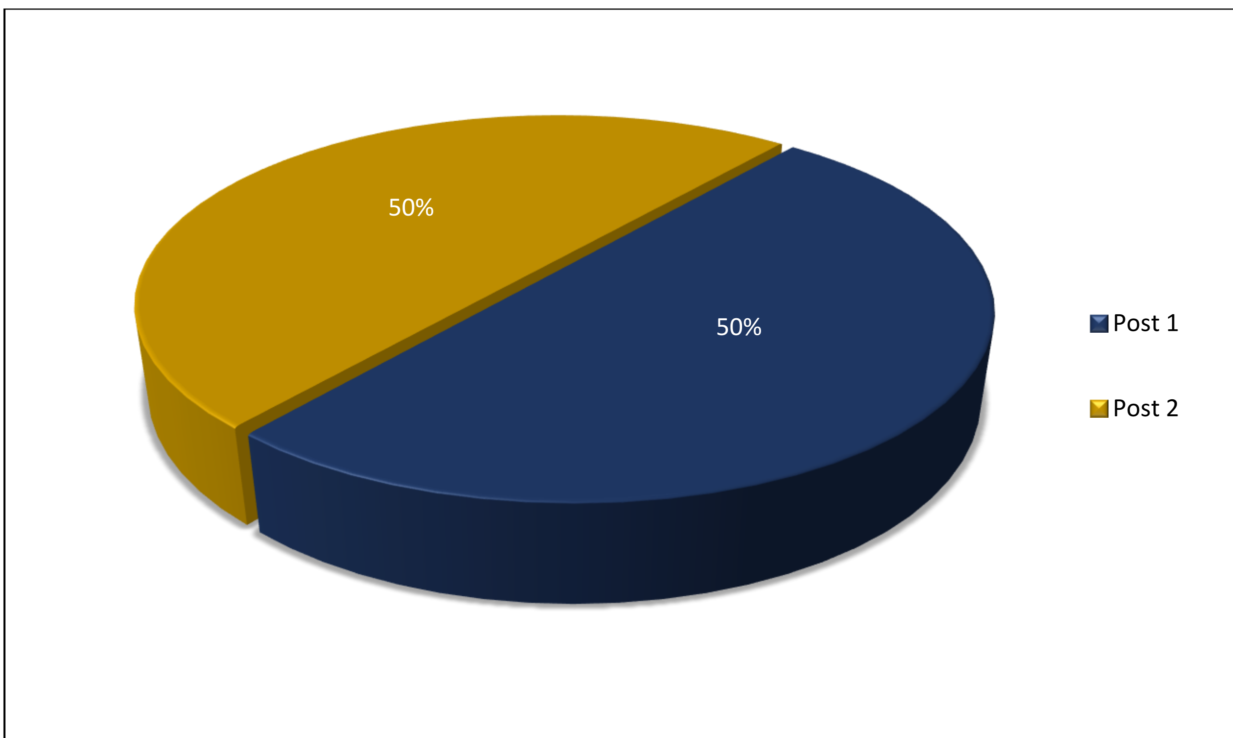


Figure 4 illustrates the distribution of the pumping capacity of each pumping station as a percentage of the total high service pumping capacity. Each of the two pumping station facilities has significant storage and ample high service pump capacities which are capable of delivering large volumes of water into the distribution system at any time to meet peak hourly and fire suppression demands.

Pumps at the two pumping stations are called on and off based on water levels in the elevated tank and local pressures. Currently, Variable Frequency Drives (VFDs) are used at both pumping stations for all pumps.

3.3 Water Storage Facilities

Water storage in Tinley Park is currently provided as both elevated and ground level storage. A summary of the water storage capacities is listed below, and a breakdown of the available storage in the Village is noted in Table 12.

TABLE 12

Available Storage Volume and Locations

Storage Type	Name/Location	Volume (MG)
Elevated Tanks	Post 11	1.0
	Subtotal	**1.0**
Ground Level	Post 1 [West]	5.0
	Post 1 [East]	5.0
	Post 2 [North]	5.0
	Post 2 [South]	5.0
	Subtotal	**20.0**
TOTAL		**21.0**

FIGURE 5

Storage by Function

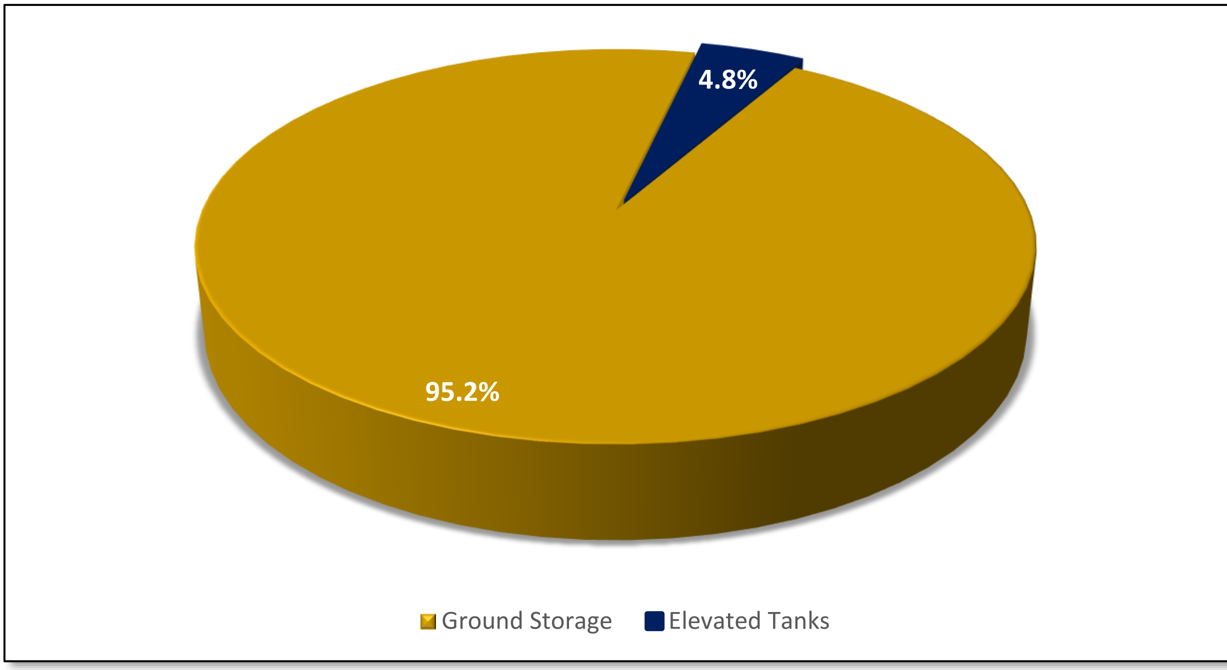
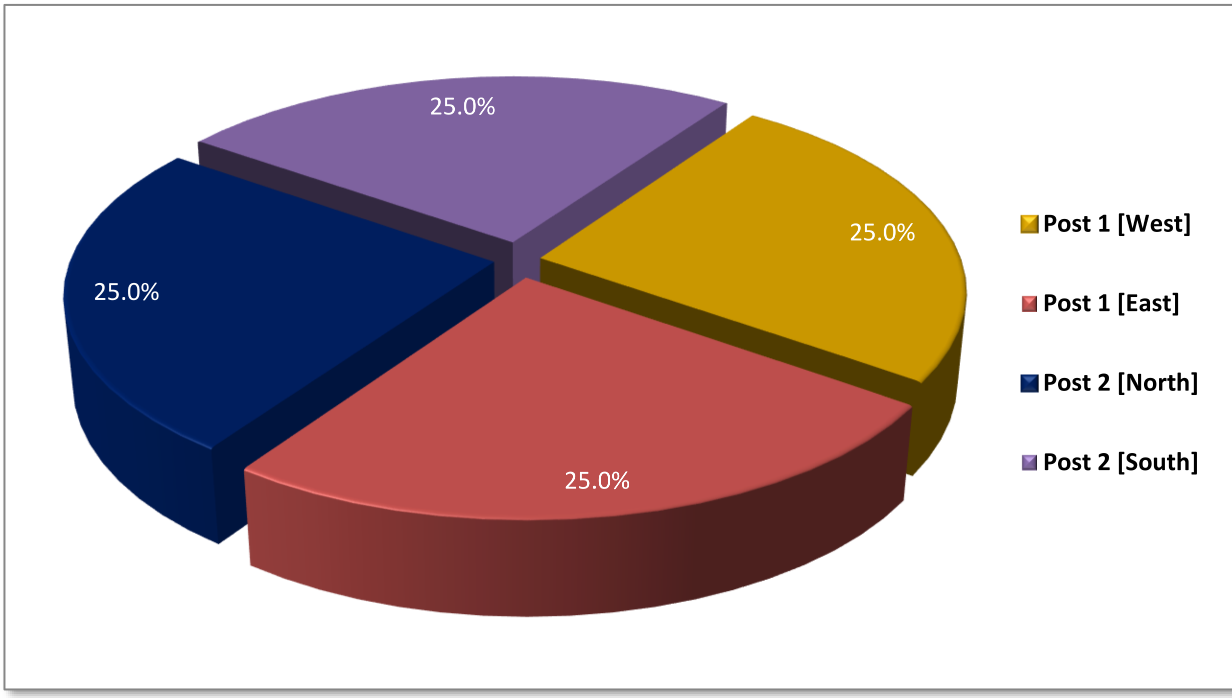


Figure 5 illustrates the percentage of elevated tank and ground level storage in relationship to the total storage volume. Currently, 4.8% of the Village's total storage volume is available through elevated tanks and 95.2% of the total storage volume is available through ground level reservoirs. Elevated storage tanks can instantly respond to changes in demand, without the use of booster pumps, controls, or electrical power. Water from ground level storage tanks must rely on pumps and controls to promptly deliver water into the distributions system. Standby power is required to allow for storage in the ground level reservoirs to be pumped into the system during power failures.

The percentage of ground storage in relationship to the total storage volume for the four ground level reservoirs is shown in Figure 6. Currently, 25% of the Village's total ground level storage volume is available at every single ground level storage reservoir.

FIGURE 6

Storage Volume at each Reservoir



3.4 Distribution System Piping

The Village's distribution system contains approximately 270 miles of water main with diameters ranging between four and thirty-six inches. A breakdown of public pipe lengths by diameter is included as Table 13.

TABLE 13

Distribution System Piping Size

Diameter (in)	Length (ft)	Length (miles)	Percent of Water Main
4	1,379	0.26	0.10%
6	404,468	76.60	28.46%
8	568,765	107.72	40.02%
10	70,120	13.28	4.93%
12	269,393	51.02	18.96%
14	55	0.01	0.00%
16	27,089	5.13	1.91%
18	102	0.02	0.01%
20	7,118	1.35	0.50%
24	55,856	10.58	3.93%

Diameter (in)	Length (ft)	Length (miles)	Percent of Water Main
30	667	0.13	0.05%
36	16,045	3.04	1.13%
Total	**1,421,057**	**269.14**	**100%**

The Village's distribution system ranges in age with mains which were constructed as part of the Village's original water system in the 1950s through 1960s, to new water mains installed in conjunction with ongoing public works projects. Approximately 70% of the Village's water main was installed from 1970 to 1990 during the Village's most rapid period of growth. A summary of pipe age is shown in Table 14.

TABLE 14

Distribution System Piping Age

Decade of Installation	Length (ft)	Length (miles)	Percent of Water Main
1950s	10,056	1.90	0.71%
1960s	98,777	18.71	6.95%
1970s	326,056	61.75	22.94%
1980s	236,378	44.77	16.63%
1990s	443,418	83.98	31.20%
2000s	281,158	53.25	19.79%
2010s	25,214	4.78	1.77%
Total	**1,421,057**	**269.14**	**100%**

The Village's Distribution System consists almost entirely of Ductile Iron pipe with a small amount of PVC pipe, as outlined in Table 15 below.

TABLE 15

Distribution System Piping Material

Pipe Material	Length (ft)	Length (miles)	Percent of Water Main
Ductile Iron	1,417,798	268.52	99.77%
PVC	3,259	0.62	0.23%
Total	**1,421,057**	**269.14**	**100%**

The life span of ductile iron and cast iron pipes can vary greatly depending on water chemistry, soil properties, installation practices, pressure fluctuations, and other factors. The primary drivers for replacement of such pipes are the need for additional capacity to meet hydraulic fire flow requirements, to transfer water to outlying areas, and high frequencies of water main breaks. Some portions of the Village's system are beginning to experience water main breaks at frequencies which warrant near term replacement. These areas are described in greater detail in Section 4.

For budgetary planning purposes, it is useful to assume a typical design life for quantifying the need for replacements. A design life of 50 years is recommended to be utilized for cast iron and ductile iron pipes built in the 1950s and 1960s based on experience with similar water systems throughout the area. A design life of 100 years is used for ductile iron pipes installed after the 1970s. A design life of 100 years is used for polyvinyl chloride (PVC) pipes.

It is worth noting that approximately 7.5% of the water system (1950s-60s pipe) has reached the end of its design life and should be replaced in the short term, this is the equivalent of roughly 20.6 miles of water main. The remaining 250 miles of public water main has a 100-year projected life span. The Village should ideally aim to replace or rehabilitate approximately 1% of the system, or 2.7 miles of water mains annually.

3.5 Water System Operations

The Village utilizes its SCADA system to control, manage, and monitor pumping operations within the water system. The SCADA system provides the water system resilience to varying conditions and operational flexibility. In general, the SCADA system functions on the basis of a "Pump Control Matrix". The matrix setup allows operators to select a sequence of pumps that turn on and off based on the elevated control tower (Post 11 Tank) and local pressures near pumping stations. The SCADA system regulates the water system based on the control matrix. For redundancy, each pumping station acts independently of the other; the control algorithm does not require nor ever utilize operational data from the other station.

Under normal operations, the Post 2 Pumping Station operates as the lead pumping station. This pumping station is located on the southeast side of the Village on 183rd Street. This pumping station has four 5.8 MGD VFD pumps that draw water from two 5 MG above-ground reservoirs. The station operates in a constant pressure mode, which means the control system turns pumps on and off and controls their speed to meet a desired pressure. The targeted pressures are grouped into stages. Each pressure stage is associated with the elevated tank level. The system is designed so that the station pressure stage increases as the elevated tank level decreases. If a single pump is running at 100% and cannot maintain its current pressure stage, an additional pump is activated.

The Post 1 Pumping Station is located on the northeast side of the Village on 167th Street. This pumping station has four 5.8 MGD VFD pumps that draw water from two 5 MG above-ground reservoirs. The station operates in a constant pressure mode just like its southern counterpart. However, the pressure stages for Post 1 are set to have this pumping station operate as the lag pumping station.

4. WATER MODEL DEVELOPMENT

4.1 General Description

The Village of Tinley Park's water system was modeled utilizing a computer program called WaterGEMS® to accurately simulate the entire water system including sources of pump input, storage facilities, and the water mains within the distribution system. The distribution model provides several advantages to the Village and was used to evaluate the following:

1. Approximate the delivery capability at any point in the system without conducting hydrant tests in the field.

2. Identify inadequate sized water mains and evaluate proposed improvements to correct these inadequacies.

3. Simulate any variety of existing or proposed facility operating scenarios and/or demand conditions.

4. Evaluate the adequacy of proposed water mains in new service areas and assess the impact the proposed new service areas will have on the existing system.

5. Help performing future water quality evaluations, such as water age analyses.

Model Description **-** The WaterGEMS® computer modeling program developed by Haestad Methods was used to build the updated water system model for this project. The model includes pipes, pipe intersections (junctions), fire hydrants, control valves, storage facilities, and pumps corresponding to the actual physical properties of the distribution system. The data required to run the computer model includes inputting various pipe characteristics (diameter, length, and hydraulic pipe friction coefficient), ground elevations throughout the service area, geographically based system water demands, storage facility characteristics (volume, geometry, and elevation) and pump characteristics (pump curves and operational controls).

4.2 Hydraulic Model Construction

The construction of the water model began by obtaining the most up-to-date distribution and facility system data from the Village's GIS. GPS data used to correct the locations of water mains, hydrants and valves. The model was created by importing data such as elevations, water main attributes, and storage tank locations from the GIS and assigning the data to the applicable model features. Additional information was added where needed to ascertain the size, length, material, location, and approximate installation year of every water main in the system. Village staff helped review the distribution system data and provided information on any missing or incorrect pipe age, material, and diameter. The Village's updates were completed in the Village's GIS and then entered into the model.

The Village provided demand data for each of the Village's water billing accounts. The billing information was processed by geocoding the account addresses to associate an approximate location with each account in the GIS. The geocoded account addresses were used to create a shapefile of point data which included the annual metered demand for each account. A module within the water model software utilized this shapefile to assign each account's demand to the nearest water main. This method of geographical demand assignment more accurately reflects how water demands vary throughout the system than other water model loading methods which have traditionally been utilized.

Ground surface elevations were assigned to each of the pipe junctions and fire hydrant locations using the elevation contours obtained from Will and Cook Counties. This provides a highly accurate indication of how pressures vary throughout the system based on ground surface elevations.

4.3 Model Calibration

The model has been calibrated to improve its overall accuracy. Calibration consisted of adjusting parameters such as pipe friction factors within the model so that predicted model results match field data obtained from the system as closely as possible. Field data used in calibration included pressures and flow rates observed during hydrant flow tests completed in the field in May of 2020, along with elevated tank water levels and pumping rates during the field testing. Hydrant flow tests were conducted to stress the system by creating large local pressure drops caused by the high hydraulic pipe friction losses associated with increased water main velocities.

Actual flow rates, which include daily demands and hydrant flows from May 2020, and measured residual pressures, were compared with the predicted flows and residual pressures from the computer model. In those cases where the actual and predicted values for flow and pressure were different, the roughness coefficients (Hazen-Williams "C" factors) of the pipes within the model were then adjusted to recreate the field results as closely as possible. Twelve of the tests fell within the targeted accuracy of plus or minus 5 psi for flow conditions. It is not uncommon to have some test locations where the targeted calibration accuracy is not achieved due to possible pipe diameter errors, incorrect connections, undocumented field changes, and the potential for closed distribution valves.

The model should be periodically recalibrated to incorporate any major distribution revisions as they are constructed and to address the impacts of any mapping anomalies or closed valves identified since the prior calibration.

4.4 Existing System Evaluation

Steady state simulations of maximum day and average day loading conditions were conducted to determine expected operating conditions under varying degrees of demand. Exhibit B shows existing pressures within the system under current average day demand periods and elevated tank levels set at typical operating levels. Exhibit C shows existing pressures under current maximum day demands and elevated tank levels set at typical operating levels.

Pressures were primarily within Ten State Standards recommended range of 35 psi minimum working pressure and 50-80 psi average working pressure. The water model demonstrates the expected maximum day pressures throughout the system range from about 29 psi to 64 psi. It is important to note that high elevation areas in the far western part of the system have pressures below 35 psi.

The available fire flow within the system varies significantly by location and water main size with in most areas meeting recommended levels of fire flow. The results of these and related analyses are discussed in greater detail within Section 5.

4.5 Water Quality/Extended Period Simulations

An extended period simulation (EPS) model was developed to simulate operational performance over a period of time. The EPS contains the same pipe and tank information as a steady state model, but includes the integration of water delivery flow rates from Oak Lawn transmission mains and to the Village's wholesale customers along with booster pump, reservoir, and elevated tank operations to simulate tank filling and withdrawal controls to recreate the Village's automated control system. Extended period simulations are very useful for identifying potential water quality concerns. Expected water age was tracked throughout the system under varying demand levels, control set points, and scenarios for a period of 4 days. The results of these analyses are discussed in greater detail within Section 5.

5. WATER SYSTEM PERFORMANCE EVALUATION

5.1 High Service Pump Evaluation

High service booster pumps are necessary to deliver water out of the ground level reservoirs to meet high demand conditions during maximum day demand periods as well as during emergencies for fighting fires or water main breaks.

As noted in Section 3, the Village has 46.1 MGD (32,000 gpm) of total pumping capacity and 34.6 MGD (24,000 gpm) of firm pumping capacity. Higher than average demands normally occur between the hours of 7:00 AM and 7:00 PM on any given day. A "conservative" estimate to determine the peak hourly rate (1.65 times the maximum rate of use) and the average delivery rate for a 12-hour period. Utilizing a peak hour demand, including wholesale exports, during the day at 1.65 times the maximum day demand, the required future design peak hourly high service pump capacity needed (ignoring any contribution from elevated storage) is 27.08 MGD (17,474 gpm). The total available long term "firm" high service booster pumping capacity of 34.6 MGD greatly exceeds the future peak hourly demands of 27.08 MGD. The Village should monitor the population growth and water use in New Lenox and Mokena.

5.2 Water Storage Evaluation

Water storage facilities provide the following: (1) water to meet the peak hourly demands, (2) water for fire protection, and (3) a reserve capacity for emergencies such as periods when the supply system is inoperable. Currently, the Village of Tinley Park has three storage facilities including one elevated tank and four reservoirs with a total capacity of 21.0 MG, as outlined in Table 12.

Several design criteria can be considered when developing overall system storage recommendations. These generally recognized design criteria include the following:

1. Two Times Average Day Storage Volume Supplier Requirement

2. One Average Day Storage Volume Recommendation (Plus Fire Suppression Needs)

3. Combined Peak Hour, Fire Flow, and Emergency Reserve Recommendations

5.2.1 Two Times Average Day Storage Volume Supplier Requirement

The Village of Tinley Park currently has an intergovernmental water supply agreement with the Village of Oak Lawn. This agreement states, "each municipal Customer shall maintain and operate, at its own cost and expense, facilities for the storage of Chicago Water sufficient in the aggregate to store not less than two (2) times its respective average day's use of water (calculated on an average annual daily basis)". For Tinley Park, the current and future "two times average day use" is the equivalent of 8.20 MG and 9.40 MG of total system storage, respectively. With 21.0 MG of existing storage, the Village has more than adequate storage to meet the "two times average day use" criterion as stated in the intergovernmental water supply agreement with the Village of Oak Lawn.

5.2.2 Average Day Storage Volume Recommendation

The IEPA commonly refers to a Ten States Standards guideline which advocates that water systems have a minimum storage volume equal to the amount of water that the system would normally deliver during "one average day demand" period. An additional amount is also normally added to the "one average day demand" volume for fire suppression needs as a guide for recommended minimum recommended storage volume. For Tinley Park, the current and future "one average day" is the equivalent of 4.10 MG and 4.70 MG of total system storage, respectively. The storage volume recommended for fire protection is dependent on the fire flow rate and duration. The maximum fire flow rate is recommended to be 3,500 gpm for a three hour duration to accommodate industrial, institutional, and commercial buildings. Fire suppression needs at 3,500 gpm for 3 hours would increase total system storage needs by 0.63 MG to 4.73 MG and 5.33 MG, respectively, as shown in Table 16. With 21.0 MG of existing storage, the Village has more than adequate storage to meet the "one average day" criterion with an allowance for additional fire protection. It should be noted that excessively large volumes of stored water can result in a decrease in water quality. Chlorine levels decrease over time as water sits in reservoirs and elevated tanks.

TABLE 16

Average Day Storage Volume

Storage Recommendations	Existing Volume (MG)	Future Volume (MG)
Average Day Demand	4.10	4.70
Fire Suppression	0.63	0.63
Total Recommended	**4.73**	**5.33**
Existing Storage Volume	21.0	21.0
Excess Storage	**16.27**	**15.67**

5.2.3 Peak Hourly, Fire Flow, and Emergency Reserve Analyses

Peak Hour - This approach estimates the amount of storage needed to provide water in excess of the average hour demands during a maximum day. For Tinley Park, this equates to a recommendation for current and future peak hour storage recommendations during design maximum day at 12.42 MG and 14.25 MG, respectively.

Fire Flow - The storage volume recommended for fire protection is dependent on the fire flow rate and duration. The maximum fire flow rate is recommended to be 3,500 gpm for a three hour duration to accommodate industrial, institutional, and commercial buildings. This is equivalent to a total of 630,000 gallons. This fire flow volume is to be added to the peak hourly demand volumes recommendation calculated above.

Emergency Supply - An additional amount of storage is also recommended to provide a reserve supply of water to meet system demands during emergencies and provide additional backup storage in the event of a major fire occurring when system storage is partially depleted. This reserve amount is typically set at 25% of total storage provided. For the purpose of this calculation, we will take the amount of combined storage estimated above (peak hour plus fire flow) and multiply by 0.25 to obtain a reserve amount equal to 25% of the total. For Tinley Park, the emergency reserve storage volume is 5.25 MG.

The current and future recommended combined peak hourly, fire flow, and emergency reserve storage volume for Tinley Park is 18.30 MG and 20.13 MG, respectively, as shown in Table 17. The Village does meet this recommendation

for emergency storage for current and future demands.

TABLE 17

Peak Hourly, Fire Flow, and Emergency Reserve Storage Volume

Storage Recommendations	Existing Volume (MG)	Future Volume (MG)
Peak Hourly Demand	12.42	14.25
Fire Flow	0.63	0.63
Emergency Supply	5.25	5.25
Total Recommended	**18.30**	**20.13**
Existing Storage Volume	21.00	21.00
Excess Storage	**2.70**	**0.87**

5.3 Water Distribution System Evaluation

5.3.1 Water System Fire Flows

Table 18 shows recommended minimum levels for fire protection based on land use classification. These recommended fire flows for residential land uses are based upon Insurance Service Office (ISO) standards for single family homes and the value corresponding to maximum credit for community water systems under the scoring for ISO's community protection ratings. The recommended fire flows shown for General Commercial, Office/Research, and Multi-family residential are interpolated based on the target for single family homes and the 3,500 gpm value corresponding to the maximum system credit. Together, the table provides a rule of thumb for desired fire protection and prioritization of distribution improvements.

The actual required fire flow for all building classifications except single family homes is a function of several attributes including building size, materials of construction, and on-site fire protection measures such as individual building sprinklers. In many cases, the calculated needed fire flow of an individual building will exceed the 3,500 gpm design fire flow for community water systems. In these cases, onsite measures should be pursued to limit the required fire flow to the distribution system capabilities. In areas of new development, it is recommended that the minimum recommended fire flow rates in Table 18 be utilized to establish minimum standards for distribution system sizing.

TABLE 18

Minimum Recommended Fire Flow Rates

Land Use	Minimum Fire Flow (gpm)
Single-Family Residential	1,000
Multi-Family Residential	2,500
Commercial – General	2,500
Office/Research	2,500
Commercial – Downtown	3,500
Institutional	3,500
General Manufacturing	3,500

When the WaterGEMS® model was run for the maximum day demand scenario, it was found that some portions of the system had inadequate fire suppression. In general, most of the areas with poor fire flow are within older parts of the system with smaller diameter mains. The lower fire flows in these smaller diameter mains is typical for communities developed in the same time frame as Tinley Park. These smaller diameter mains were not originally designed to deliver the modern day required fire flow. In addition, there are some fire flow issues near facilities with increased fire flow requirements, such as schools, that are located within primarily residential areas with inadequately sized mains. The maximum day fire flows are illustrated in Exhibit D. Exhibit E displays the fire flow deficiencies that are less than the flow recommended for a particular land use classification.

Improvements to fire flows can be made with a combination of distribution and other facility improvements. These improvements are discussed in more detail in Section 6, after a review of other water main and distribution issues.

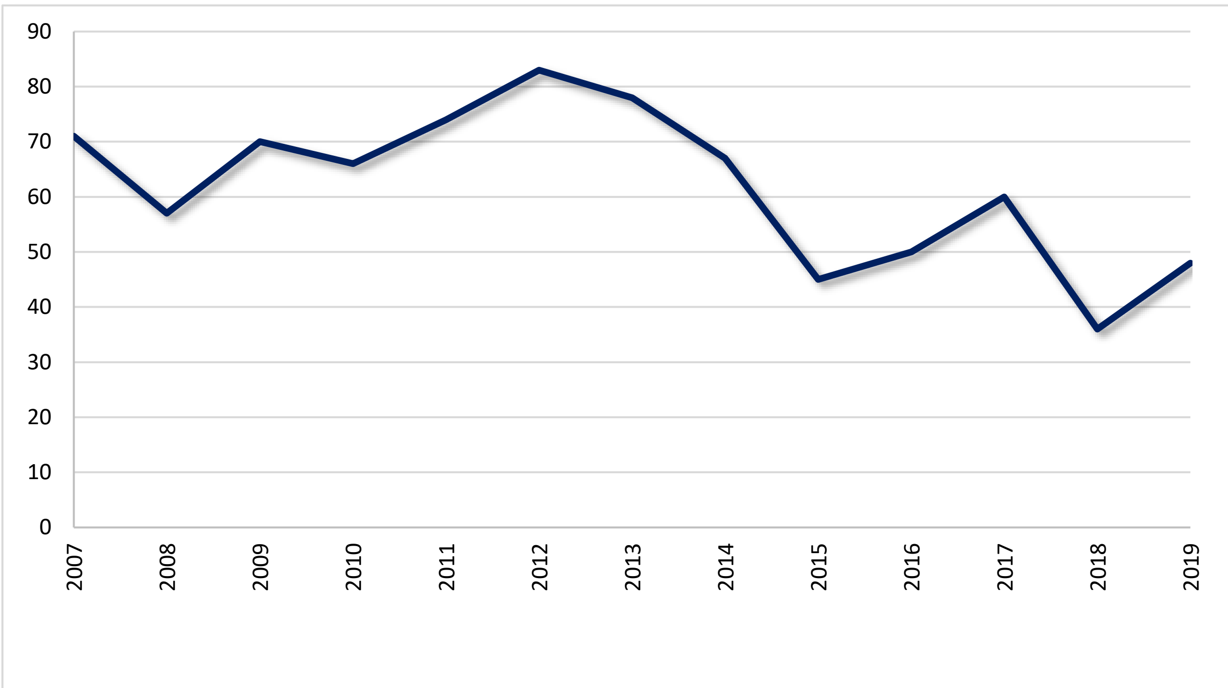
5.3.2 Water Main Breaks

Exhibit F shows the location of main breaks in terms of the number of breaks per pipe segment. From 2007 through 2019, there were 803 water main breaks with an average of 62 breaks per year during that span (excluding 2020). According to Village records, the average cost of a water main break is $2,300. With an annual average of 62 breaks, the Village is spending more than $143,000 each year on water main break repairs.

The water main breakage rate in Tinley Park is less than the 25 main breaks per 100 miles rate considered reasonably acceptable by the American Water Works Association (AWWA). Main breaks cause temporary water quality deterioration, service disruptions for customers, pose a risk to the field crews entering excavations to repair breaks, cost money to repair in terms of labor and materials, and force the Village to spend additional money as the lost water must be repumped and treated. A summary of Tinley Park's annual water main breaks is shown in Figure 7.

FIGURE 7

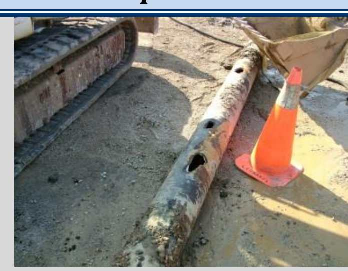
Main Break Count



Water main breaks typically fall into one of three categories as outlined in Table 19 below.

TABLE 19

Main Break Types

Main Break Type	Failure Description	Typical Causes of Failure	Sample Photos
Blow Out	Hole in pipe with defined shape and area	Isolated point of pressure or corrosion	

Main Break Type	Failure Description	Typical Causes of Failure	Sample Photos
Stress/Shear	Circular break around perimeter of the water main	Poor installation, frost heave, inadequate pipe support, water hammer	
Lateral	Elongated main break	Stress caused by inadequate pipe bedding	

Other miscellaneous main break types include "pinhole" leaks caused by point corrosion and pipe joint failures generally caused by improper installation. There are multiple factors that are contributing to all of Tinley Park's breaks which include the following:

Pipe Age: Age is not necessarily an indicator of anticipated water main breaks, but age can be significant when considered with other factors such as local corrosion rates and the type of pipe material. It has been our experience that ductile iron and spun cast iron pipes manufactured and installed in the 1950s and 1960s have a tendency to corrode and fail prematurely. As shown in Table 20, the highest frequency of the main breaks occur in the 1950-60s main. Break frequency was calculated by dividing the total number of breaks occurring on water main installed in a particular decade by the total length of main installed in that decade. It is presumed that the Village grew rapidly during the 1960s and it is highly possible that this main was not installed with current best management practices.

TABLE 20

Number of Breaks per Decade of Installation

Decade of Installation	Number of Breaks	Break Frequency [breaks/1000 ft]	Percentage of Total Breaks
1950	20	1.99	2.5%
1960	230	2.33	28.6%
1970	291	0.89	36.2%
1980	96	0.41	12.0%

Decade of Installation	Number of Breaks	Break Frequency [breaks/1000 ft]	Percentage of Total Breaks
1990	87	0.20	10.8%
2000	48	0.17	6.0%
2010	31	1.23	3.9%
Total	**803**	**7.2**	**100.0%**

Pipe Size: Smaller mains generally exhibit higher failure rates than larger diameter mains as shown in Table 21. The reasons small diameter mains are more vulnerable are because they have a smaller cross sectional area and have wall thicknesses that are thinner than those of larger pipes. These characteristics make the smaller pipes more vulnerable to beam failure and corrosion.

TABLE 21

Number of Breaks per Pipe Diameter

Diameter (in)	Number of Breaks	Break Frequency [breaks/1000 ft]	Percentage of Total Breaks
4	0	0.00	0.0%
6	483	1.19	60.1%
8	244	0.43	30.4%
10	29	0.41	3.6%
12	39	0.14	4.9%
14	0	0.00	0.0%
16	2	0.07	0.2%
18	0	0.00	0.0%
20	0	0.00	0.0%
24	5	0.09	0.6%
30	1	1.50	0.1%
36	0	0.00	0.0%
Total	**803**	**3.84**	**100.0%**

Pipe Material: Common water main piping materials include cast iron, ductile iron (DI), asbestos cement (also called transite), polyvinyl chloride (PVC), high density polyethylene, and prestressed concrete cylinder pipe. Varying material characteristics such as wall thickness, corrosion resistance, and flexibility all factor into water main breaks.

Table 22 shows that nearly 100% of the water mains breaks experienced are on ductile iron mains. Additionally, a large percentage of main breaks occur on ductile iron pipe mainly installed in the 1950s and 1960s, during a rapid growth period for the Village. It has been our experience that ductile iron and spun cast iron pipes manufactured and installed in the 1950s and 1960s have a tendency to corrode and fail prematurely in the Chicagoland area.

TABLE 22

Number of Breaks per Pipe Material

Pipe Material	Number of Breaks	Break Frequency [breaks/1000 ft]	Percentage of Total Breaks
Ductile Iron	801	0.56	99.8%
PVC	2	0.61	0.2%
Total	**803**	**1.20**	**100.0%**

Temperature: Main break occurrences can be related to temperature. Freezing temperatures penetrate soil and pavement above water mains and create soil and pipe stresses due to expansion and contraction of the soil and pipe material. Cold Lake Michigan water can also make cast and ductile iron pipe more brittle in some systems. Drought conditions, such as in the summer of 2012, also lead to main breaks. The ground pulls away from the main leaving it more susceptible to movement and resulting in increased stress on the water main.

Pressure Differentials: If the water pressure in a system frequently fluctuates over a substantial range of pressures, it will have a fatiguing effect on the pipe as it expands and contracts in response.

Installation and Location: Water mains installed in heavily traveled areas can be more susceptible to breaks due to increased vibrations and loadings. Water mains located in proximity to construction activities can also be vulnerable due to subsurface disturbances and increased loadings from construction equipment. The installation methods themselves can also contribute to water main breaks. It is theorized that a combination of early application ductile iron pipe produced in the 1960s, in combination with a lack of pipe bedding during the same period, has contributed to premature failure of these pipes.

Soil Type: Some soils create a corrosive environment for cast iron, ductile iron, and steel water main pipe. For example, silty clay loam is generally poorly to moderately drained and generally classified as corrosive to metallic pipes.

5.3.3 Water Main Break Ranking

To assist with the prioritization of water main replacements, the mains with a history of main breaks have been ranked using measurable parameters. This task has been accomplished using the Tinley Park GIS system in collaboration with the water model and a water main break ranking equation.

The ranking equation is based on selection of those factors that best define the expectation that a particular water main segment will develop a leak and the potential impact of that leak on the distribution system and customers as a whole. The equation considers the relative importance and applicability of the influencing factors described above, as well as the historical frequency of main breaks per segment, the length and size of each segment, water flow volume, and head loss. Mains with breaks that have been replaced in the past 12 years have been removed from the ranking process.

Four key factors have been calculated for every pipe segment in the water system that has experienced a main break and the resulting values inserted into the following basic ranking equation:

*TOTAL RANK = (0.4 * Water Main Break Frequency Rank) +*
*(0.15 * Flow Volume Rank) + (0.15 * Head Loss Rank) + (0.3 * Remaining Life)*

The four factors were selected based on the discussion below. The percentage of criticality of each of the four key factors is represented as 40% for Water Main Break Frequency, 15% for Flow Volume Rank, 15% for Head Loss Rank, and 30% for the pipe's remaining life span. The criticality factors are based on discussions with Village staff and best practices in the water system industry.

The "Water Main Break Frequency Rank" is based on the number of main breaks per 1,000 lineal feet since 2007. Pipe segments are first sorted from highest to lowest based on the number of breaks per 1,000 feet and the percentile ranking of that segment relative to the other mains (top ten percent). The percentile value is then multiplied by 10 and rounded to the nearest whole number. This means of calculation results in approximately the highest five percent of pipes with breaks being assigned a rank of "10"; the lowest five percent assigned a rank of "0"; and the remaining pipe segments evenly assigned to each ranking value between "1" and "9", inclusive.

The "Flow Volume Rank" indicates which pipe segments have the most hydraulic importance to the system. A main break on a large water main conveying a large volume of water will impact a greater portion of the system than will smaller, low flow segments. The Flow Volume Rank is based on each segment's flow relative to the maximum observed pipe flow. Pipe segments are first sorted from highest to lowest based on their flow and the percentile ranking of that segment relative to the other mains. The percentile value is then multiplied by 10 and rounded to the nearest whole number. This calculation results in approximately the highest five percent being assigned a rank of "10"; the lowest five percent assigned a rank of "0"; and the remaining pipe segments evenly assigned to each Flow Volume Rank between "1" and "9", inclusive.

The "Head Loss Rank" is based on friction head loss (in feet) per 1,000 lineal feet of pipe. This variable indicates whether a pipe has adequate cross-sectional area to meet flow requirements. Increased head loss occurs due to build-up of deposits in the pipe, inadequate pipe size, or obstructions present in the pipe. The model uses the same maximum day demand simulation as for the Flow Volume Rank to calculate a friction head loss value for each pipe segment. Pipe segments are first sorted from highest to lowest based on their head loss and the percentile ranking of that segment relative to the other mains. The percentile value is then multiplied by 10 and rounded to the nearest whole number. This calculation results in approximately the highest five percent being assigned a rank of "10"; the lowest five percent assigned a rank of "0"; and the remaining pipe segments evenly assigned to each Head Loss Rank between "1" and "9", inclusive.

The “Remaining Life Rank” is based on the expected remaining life of the pipe segment. We estimated remaining life based on the following criteria: Estimated PVC Pipe Life is 100 years, Estimated DI Pipe built in the 1950s and 1960s is 50 years, Estimated DI Pipe built in the 1970s and on is 100 years, Estimated Cast Iron Pipe life is 85 years, and Estimated Asbestos Cement Pipe Life is 70 years. Remaining life was calculated for each pipe segment in the Village based on the pipe material and install date. The percentile value is then multiplied by 10 and rounded to the nearest whole number. This calculation results in approximately the highest five percent being assigned a rank of “10”; the lowest five percent assigned a rank of “0”; and the remaining pipe segments evenly assigned to each Head Loss Rank between “1” and “9”, inclusive.

The four ranking factors are then multiplied by their assigned factors as shown in the formula and summed to provide the total rank for each water main segment. The maximum possible segment rank is a value of 10 with the adjustment factors shown above.

All segment numbers and their corresponding ranking ranges are then displayed within the GIS system as shown in Exhibit G. This ranking should then be considered in conjunction with the fire flow findings and other Village capital projects as a means of setting the final pipe replacement prioritizations.

5.3.4 Water Loss/Non-Revenue Water

Water loss, also known as non-revenue water, is the difference between the volume of water purchased and the volume of water billed to customers. Non-revenue water is comprised of three elements: real losses, apparent losses, and unbilled authorized consumption. Real losses are the physical losses of the system, i.e., leakage from mains, storage tank overflows, service line leakage. Apparent losses are water that reaches a user but is not properly measured or paid for, i.e. customer meter inaccuracies, data handling errors, theft. Unbilled authorized consumption includes system flushing and testing, construction, government buildings, etc.

The Illinois Department of Natural Resources (IDNR) has finalized the amended Lake Michigan Allocation rules for communities, such as Tinley Park, that receive Lake Michigan water. The revised standards allowed for a non-revenue water limit of 12% in 2015, and decreased to 10% in 2019. The Village’s Non-Revenue water in 2019 was calculated at 10.0%.

The Village has been dedicated to addressing Non-Revenue water over the years. The Village annually completes leak detection of the system, has a meter testing program in place, and is dedicated to addressing main breaks in a timely manner.

5.3.5 Water Main Repair and Rehabilitation Considerations

Several areas of the older residential neighborhoods have water mains that have less than recommended fire flows and also have high water main break history. These locations may be candidates for replacement versus repair. A more detailed discussion of potential replacement and repair options is presented below.

In-Situ Replacement - Methodologies we considered include: micro-tunneling; pipe bursting; horizontal directional drilling; cement lining of iron pipes; installing semi-rigid liners; and cured-in-place liners. Associated costs with various water main improvement options are shown in the table below. The rehabilitation options are compared to an estimated standard cost for open cut replacement of an 8-inch diameter water main, versus typical range of costs for various pipe rehabilitation or in-situ replacement techniques. Prices include water service replacement, new valves, new hydrants, trench backfill, and trench pavement or lawn restoration. It is important to keep in mind that every water main replacement or rehabilitation is site specific and the cost should be calculated on the known conditions; however, Table 23 provides a frame of reference for the costs variations.

TABLE 23

Pipe Improvement Methods and Associated Costs

Pipe Improvement Construction Methods	Cost Multiplier Range		Cost Per Foot Example ($/ft.)	
	Low	High	Low	High
Open-cut replacement (8-inch in pavement)	1.0		$325	
Micro-tunneling	5.0	20.0	$1,250	$5,000
Pipe bursting	0.8	1.6	$ 200	$ 400
Horizontal directional drilling	0.7	1.3	$ 175	$ 325
Semi-rigid liners	0.7	1.2	$ 175	$ 300
Slip-Lining	0.7	1.2	$ 175	$ 300
Cement lining of iron pipes	0.6	0.9	$ 150	$ 225
Cured-in-place liners	0.6	0.8	$ 150	$ 200

Based on our review of the available pipe rehabilitation and in-situ replacement techniques and the condition of the existing pipes, it is our opinion that there may be some instances that pipe rehabilitation or in-situ replacement will be more cost effective and practical than traditional open-cut construction. However, in most Tinley Park locations, open cut replacement will remain the most cost effective means to improve fire flow and reduce water main breakage. We recommend the Village consider these methodologies on a case-by-case basis as a replacement for open-cut construction with each project, but that preliminary project budgets be based on the estimated cost of conventional open-cut installation.

5.3.6 Water Age

It is important for a distribution system to circulate its water in order to reduce the amount of stagnant water and maintain adequate chlorine residual. Inadequate chlorine residual levels tend to start to occur when water age is greater than five days. Average water age can be lowered throughout a system by providing adequate mixing and turnover in storage facilities.

Extended period simulations were run to simulate existing demand and operating conditions. In conversations with Village staff it was estimated influent water age at both Oak Lawn Supply Points is 24 hours. Water age after 4 days of simulation during average day conditions is shown in Exhibit H. Results of the extended period simulation modeling show that, as expected, the extreme edges of the system show the highest expected water age due to the distance from the reservoir pumping stations. The maximum water age seen in the simulations is approximately 117.8 hours, or 4.9 days.

5.3.7 Proposed West Pressure Zone

Currently, the Village is experiencing substandard pressures (<35 psi) in the west part of the system near S La Grange Road. The Village is currently evaluating the feasibility of implementing a new West Pressure Zone by constructing a booster station and three pressure reducing valve (PRV) stations to raise the hydraulic grade line (HGL) and increase pressures in the western part of the system. Table 24 and Exhibit O summarizes the recommended facility locations that were simulated within the water model.

TABLE 24

Potential West Pressure Zone Facility Locations

Proposed Facility	Proposed Facility Location
West Pressure Zone Pump Station	179th Street & 88th Avenue
12" PRV Station #1	171st Street & 88th Avenue
12" PRV Station #2	175th Street & 88th Avenue
12" PRV Station #3	183rd Street & 88th Avenue

Steady state simulations were used to evaluate the feasibility of the potential West Pressure Zone Pump Station and PRV Stations. In these scenarios, the proposed West Pressure Zone Pump Station was equipped with two 750 gpm booster pumps that can supply the demands of the proposed pressure zone while boosting pressures 20 psi. Due to the lack of elevated storage in this area, the pumps were modeled as VFDs and programmed to maintain a set discharge pressure to provide adequate pressures (>35 psi) in the West Pressure Zone.

On its own, the proposed booster station will not be able to supply enough water for a large fire flow event in the proposed west pressure zone. PRVS were placed on the large diameter mains to the north and south of the proposed pump station. The PRVs sense if pressures downstream drop below a set point and the PRVs open to provide supply pressure/flow. The PRVs were set to open and allow water from the east to flow freely to accommodate fire flow events of any magnitude. Pressures and fire flows with the proposed West Pressure Zone improvements in place are shown in Exhibit O and P, respectively.

Extended period simulations utilizing the recommended controls evaluate the change in water age with the proposed West Pressure Zone Pump Station and PRV Stations. The water age results for these simulations are shown in Exhibit Q. Exhibit Q shows a marginal increase in water age on the north and south ends of the proposed West Pressure Zone (near proposed PRV Stations), and a decrease in water age near the proposed West Pressure Zone Pump Station. The increase in water age near the proposed PRV stations can be attributed to the restriction of flow that would normally be occurring in the large diameter mains in those areas. The decrease in water age near the proposed West Pressure Zone Pump Station can be attributed to the concentration and increase in flow due to the proposed pump station.

The proposed West Pressure Zone and accompanying facilities provide Tinley Park with adequate pressure (>35 psi) and water supply during maximum day demand and emergency situations. Proposed West Pressure Zone improvements are discussed in further detail in Section 6. A hydraulic profile for the Proposed West Pressure Zone is detailed in Appendix B.

5.4 Existing and Impending Regulations

5.4.1 Existing Regulations

IEPA regulations mostly govern water system planning and design, and they set numerical water quality limits that drinking water must meet. AWWA provides a wide range of industry standards and best practices to maintain the reliability and redundancy of water systems to prolong their useful life and maximize reliability in the most cost-effective manner. Table 25 summarizes pertinent IEPA regulations and AWWA guidelines for water system operation and maintenance that relate directly to Tinley Park.

TABLE 25

IEPA Regulations and AWWA Guidelines

Category – Regulation/Guideline	Citation [1, 2]
Distribution System	
Minimum pressure 35 psi (20 psi during emergencies)	Title 35, Part 653.106
Maximum pressure 100 psi	Title 35, Part 653.106
Minimum water main size 4”	Title 35, Part 653.117
Minimum free chlorine residual 0.5 mg/L	Title 35, Part 653.604
Minimum combined chlorine 1.0 mg/L	Title 35, Part 653.604
Water Storage Tanks	
Pump suction wells shall be watertight	RSWW, Part 6.2.1
Minimum distribution storage = average daily demand	RSWW, Part 7.0.1
Finished water storage tanks must have watertight roofs that birds, animals, insects, and excessive dust	RSWW, Part 7.0.3

1 Title 35 refers to the Illinois Administrative Code, Title 35, Subtitle F, Chapter II, Part 653.
2 RSWW refers to the Recommended Standards for Water Works, 2007 Edition.

Pumping	
Secondary power supply recommended for facilities that supply water to the system by pumping from a ground storage tank	RSWW, Part 6.6.6 and AWWA M31
Firm pumping capacity must be sized to meet the Maximum Day Demand	RSWW, Part 6.3

It is important to note Illinois has recently changed the minimum chlorine residual from 0.5 mg/l to 1.0 mg/l. The Village of Tinley Park currently adds chlorine before supplying water to their customers.

6. WATER SYSTEM IMPROVEMENT; RECOMMENDATIONS AND COSTS

This section provides overall recommendations and costs for proposed major water facility and distribution improvements.

The Village currently has a number of facility maintenance and water main replacement projects budgeted through 2025. This report and associated Capital Improvement Plan (CIP) contains these budgeted items, along with a list of proposed future or long term projects. The future projects are not planned out in a yearly CIP. Rather, they are listed in order of priority to give the Village flexibility in determining yearly improvement projects based on available funding and other capital improvement projects. This report does not, however, specifically identify all replacements for the facilities or distribution system including undersized mains, older water mains, or mains with high break history.

Costs presented in this report are based on current year (2021) construction based on past projects with a factor added for engineering, legal, and unknown project requirements at this time. Costs for all projects should be verified during the preliminary design stage of a project in advance of bidding, after consideration of more detailed pilot study results, field survey reconnaissance, and other more detailed information. A spreadsheet of proposed improvement projects and the associated costs are shown in Appendix C.

6.1 Conclusions and Recommendations

6.1.1 Water System Booster Capacity

As noted in Section 3, the Village currently has 46.1 MGD (32,000 gpm) of total pumping capacity and 35.6 MGD (24,000 gpm) of firm pumping capacity. Utilizing a peak hour demand during the day at 1.65 times the future maximum day demand projections in Table 9, we calculate required future design peak hourly high service pump capacity needs within Tinley Park (ignoring any contribution from elevated storage) of 27.08 MGD. The total available long term "firm" high service booster pumping capacity of 35.6 MGD greatly exceeds the peak future hourly demands of 27.08 MGD, and no booster capacity improvements are recommended. The Village should monitor the population growth and water use in New Lenox and Mokena.

6.1.2 Water System Storage

Currently, the Village of Tinley Park has three storage facilities including one elevated tank and four reservoirs with a total capacity of 21.0 MG, as previously outlined in Table 12. Several design criteria can be considered when developing overall system storage recommendations, as described in Section 5.2. The more conservative of these approaches, the Combined Peak Hour, Fire Flow, and Emergency Reserve Recommendations method, was used to evaluate storage within the Village.

The recommended future combined peak hourly, fire flow, and emergency reserve storage volume for Tinley Park projected as 20.13 MG, as shown in Table 26. The Village does have adequate emergency storage for future demands. As a nearly fully-developed community with minimal potential for increased future storage requirements, no storage improvements are recommended for the Village.

TABLE 26

Future Peak Hourly, Fire Flow, and Emergency Reserve Storage Volume

Storage Recommendations	Volume (MG)
Future Peak Hourly Demand	14.25
Fire Flow	0.63
Emergency Supply	5.25
Total Recommended	**20.13**
Existing Storage Volume	21.00
Excess Storage	**0.87**

6.1.3 Water Distribution System

The Village currently has a number of water main replacement projects budgeted or in construction from 2021 to 2025. While these improvements are included in the water model and Appendix C, this report does not modify the Village's existing water main replacement projects. This report identifies a Capital Improvement Plan for water main improvement projects for the near term (2021-2025) and long term. The improvements are shown in Exhibit I and Exhibit L.

Improvements were identified to increase fire flows, improve service to critical users, address main breaks, and to address future developments. These improvements are noted below in Table 27 and Table 28. Projects proposed for 2021 through 2025 were coordinated with the current CIP.

TABLE 27

Distribution System Improvements (2021-2025)

	Proposed Improvements	Low Fire Flow	Connectivity & Looping	Main Breaks	Main Break Ranking	Future Development	Critical Care User
2021	La Grange Rd Utilities Extension	X	X			X	
	Proposed West Pressure Zone					X	
	S La Grange Rd & 175th St	X	X				X
	La Grange Road Gap	X	X				
2022	67th Court – 175th St to 174th St	X		X	7		
	66th Ct & 173rd Pl	X		X	7		
	Dorothy Ln			X	8		

	Proposed Improvements	Low Fire Flow	Connectivity & Looping	Main Breaks	Main Break Ranking	Future Development	Critical Care User
2023	179th St & Oak Park Ave	X		X	8		X
	174th Pl - Oak Park Ave & 66th Ave	X		X	7		
	Ironwood Drive			X	5		
2024	Terrace Dr & Skyline Dr	X	X	X	6		
	160th Place			X	9		
	Bremen Grove Loop	X	X				X
2025	176th Street			X	9		
	Ozark Ave & 159th Pl	X					
	176th Place			X	7		

TABLE 28

Distribution System Improvements (Long Term)

	Proposed Improvements	Low Fire Flow	Connectivity & Looping	Main Breaks	Main Break Ranking	Future Development	Critical Care User
Long Term	172nd St & 66th Ct	X					X
	Beverly Ave, Honey Ln, & Carlsbad Dr			X	8		
	175th St & Sandalwood Dr	X					X
	Oak Park Avenue	X					X
	Pine Ridge Dr & Misty Pines Ct	X	X				
	Apple Ln & Steven Pl	X					X
	Brementowne Dr & Sussex Rd	X					
	167th St & Gentry Ln	X					
	Ridgeland Avenue	X		X	8		
	Oak Forest Avenue			X	9		X
	163rd Street			X	9		
	70th Ave & 177th St			X	9		
	175th Street			X	9		
	Royal Oak & 180th St	X		X	7		
	163rd Place			X	8		
	164th Street			X	8		
	170th Place			X	8		
	173rd Pl, Odell Ave, & Oconto Ave			X	9		
	71st Ave & 174th Pl			X	8		
	Highland Avenue			X	8		

	Proposed Improvements	Low Fire Flow	Connectivity & Looping	Main Breaks	Main Break Ranking	Future Development	Critical Care User
	161st Place			X	7		
	164th Place			X	7		
	174th St & Osceola Ave			X	7		
	Overhill Ave & 173rd Pl			X	7		
	68th Court			X	7		
	65th Avenue			X	7		
	182nd Street			X	7		
	182nd Place			X	7		

Forty-eight near and long term distribution system improvements were developed and simulated with the model to address system deficiencies. Table 29 and Table 30 detail these improvements, including budgeted year, length, and estimated project costs (noted in Appendix C). Each year includes an allowance to address miscellaneous water main improvements needed to address low fire flow areas, water main breaks, future developments, and coordinate with Public Works projects.

TABLE 29

Distribution System Improvements (2021-2025)

Budget Year	Proposed Improvements	Proposed Diameter (in)	Length (ft)	Estimated Construction Cost	Estimated Engineering Cost	Estimated Total Project Cost
2021	La Grange Rd Utilities Extension	12	2,700	$920,000	$143,500	$1,060,000
	Proposed West Pressure Zone	-	-	$1,680,000	$370,000	$2,070,000
	S La Grange Rd & 175th St	12	2,000	$750,000	$112,000	$860,000
	La Grange Road Gap	12	600	$225,000	$34,000	$260,000
	2021 SUBTOTAL			**$3,575,000**	**$659,500**	**$4,250,000**
2022	67th Court - 175th St to 174th St	12	1,100	$410,000	$62,000	$470,000
	66th Ct & 173rd Pl	8	700	$230,000	$34,000	$260,000
	Dorothy Ln	8	990	$390,000	$58,000	$450,000
	2022 SUBTOTAL			**$1,030,000**	**$154,000**	**$1,180,000**
2023	179th St & Oak Park Ave	8	1,000	$330,000	$50,000	$380,000
	174th Pl - Oak Park Ave & 66th Ave	8	1,100	$360,000	$54,000	$410,000
	Ironwood Drive	8	1,400	$460,000	$70,000	$530,000
	2023 SUBTOTAL			**$1,150,000**	**$174,000**	**$1,320,000**
2024	Terrace Dr & Skyline Dr	8	1,400	$460,000	$70,000	$530,000
	160th Place	8	1,200	$390,000	$58,000	$450,000
	Bremen Grove Loop	12	900	$340,000	$52,000	$390,000
	2024 SUBTOTAL			**$1,190,000**	**$180,000**	**$1,370,000**

Budget Year	Proposed Improvements	Proposed Diameter (in)	Length (ft)	Estimated Construction Cost	Estimated Engineering Cost	Estimated Total Project Cost
2025	176th Street	8	1,300	$420,000	$64,000	$480,000
	Ozark Ave & 159th Pl	8	1,200	$390,000	$58,000	$450,000
	176th Place	8	900	$290,000	$44,000	$330,000
	2025 SUBTOTAL			**$1,100,000**	**$166,000**	**$1,260,000**
	2021-2025 CAPITAL IMPROVEMENTS PLAN TOTAL			**$8,045,000**	**$1,333,500**	**$9,380,000**

Exhibit I shows the project locations listed in Table 29 from 2021 through 2025. Exhibit J and K display the available fire flows and additional fire flow needed after the improvements listed in Table 29 are constructed. Exhibit K shows the decrease in additional fire flows needed near the improvements that address low fire flow areas in Table 27.

TABLE 30

Distribution System Improvements (Long Term)

Proposed Improvements	Proposed Diameter (in)	Length (ft)	Estimated Construction Cost	Estimated Engineering Cost	Estimated Total Project Cost
172nd St & 66th Ct	12	900	$340,000	$52,000	$390,000
Beverly Ave, Honey Ln, & Carlsbad Dr	8	2,500	$820,000	$124,000	$940,000
Andres Avenue	8	700	$230,000	$34,000	$260,000
175th St & Sandalwood Dr	8	1,100	$360,000	$54,000	$410,000
Oak Park Ave	8	1,300	$420,000	$64,000	$480,000
Pine Ridge Dr & Misty Pines Ct	8	300	$100,000	$16,000	$120,000
Apple Ln & Steven Pl	8	1,900	$620,000	$94,000	$710,000
Brementowne Dr & Sussex Rd	8	1,300	$420,000	$64,000	$480,000
167th St & Gentry Ln	12	2,500	$940,000	$142,000	$1,080,000
Ridgeland Avenue	12	2,100	$790,000	$118,000	$910,000
Oak Forest Avenue	12	600	$230,000	$34,000	$260,000
163rd Street	8	1,200	$390,000	$58,000	$450,000
70th Ave & 177th St	8	2,200	$720,000	$108,000	$830,000
175th Street	8	1,200	$390,000	$58,000	$450,000
Royal Oak & 180th St	8	1,300	$420,000	$64,000	$480,000
163rd Place	8	900	$290,000	$44,000	$330,000
164th Street	8	600	$200,000	$30,000	$230,000
170th Place	8	900	$290,000	$44,000	$330,000
173rd Pl, Odell Ave, & Oconto Ave	8	3,100	$1,010,000	$152,000	$1,160,000
71st Ave & 174th Pl	8	1,000	$330,000	$50,000	$380,000
Highland Avenue	8	1,300	$420,000	$64,000	$480,000
161st Place	8	2,400	$780,000	$118,000	$900,000
164th Place	8	1,000	$330,000	$50,000	$380,000
174th St & Osceola Ave	8	1,200	$390,000	$58,000	$450,000
Overhill Ave & 173rd Pl	8	1,400	$460,000	$70,000	$530,000
68th Court	8	1,200	$390,000	$58,000	$450,000
65th Avenue	8	1,000	$330,000	$50,000	$380,000
182nd Street	8	700	$230,000	$34,000	$260,000
182nd Place	8	700	$230,000	$34,000	$260,000
Harlem Ave & Vollmer Rd Development	12	5,800	$2,200,000	$330,000	$2,530,000
State Mental Health Property	12	1,900	$700,000	$106,000	$810,000
LONG-TERM CAPITAL IMPROVEMENTS PLAN TOTAL			**$15,770,000**	**$2,376,000**	**$18,110,000**

Exhibit L shows the long-term project locations listed in Table 30. Exhibits M and N display the available fire flows and additional fire flow needed after the improvements listed in Table 30 are constructed, with the exception of the Proposed West Pressure Zone Improvements. Exhibit N shows the decrease in additional fire flows needed near the improvements that address low fire flow areas in Table 28. All projects included in Table 29 and Table 30 are described in Appendix C in greater detail.

6.1.4 Proposed West Pressure Zone

The pressures and available fire flows under max day demands with the Proposed West Pressure Zone Improvements are shown in Exhibit O and P, respectively. Additionally, the estimated water age under average day demands with the addition of the Proposed West Pressure Zone Improvements is displayed in Exhibit Q.

The Engineers Opinion of Probable Cost (EOPC) for the Proposed West Pressure Zone Improvements is detailed in Table 31.

TABLE 31

Proposed West Pressure Zone Improvements EOPC

Item		Quantity	Unit	Unit Cost	Labor	Capital Cost
Excavation, backfill, and rough grade		1	LS		$18,000	$18,000
Concrete Base		4.5	CF	$2,500		$12,000
Above-Grade Prefab. Pump Station		1	LS	$295,000	$44,000	$339,000
12" PRV Station with Vault		3	LS	$131,750	$20,000	$456,000
Electrical	20%		LS	$235,000		$235,000
General Conditions	15%		LS			$124,000
					Subtotal	**$1,180,000**
				Contractor OH&P, Bonds & Insurance	**18%**	**$212,000**
					Total Construction Cost	**$1,400,000**
				Contingency	**20%**	**$280,000**
					Total Construction Cost with Contingency	**$1,680,000**
				Design Engineering	**11%**	**$185,000**
				Construction Engineering	**11%**	**$185,000**
				Legal & Administrative	**1%**	**$16,800**
					PROPOSED WEST PRESSURE ZONE IMPROVEMENTS TOTAL	**$2,070,000**

6.1.5 Future Considerations

When considering future improvements, the Village should consider water main break history, upcoming roadway improvements, and upcoming sewer improvement projects. All proposed long term future improvements were given a priority rating in order to both aid in determining water main improvement schedules, as well as allowing for flexibility with available funding, upcoming roadway or sewer improvements, or recent main break trends.

6.2 Funding Opportunities

The IEPA revolving loan program is a possible funding source for many communities. While no grant contributions are anticipated in the near future, the IEPA interest loan rate is projected to remain near 2%.

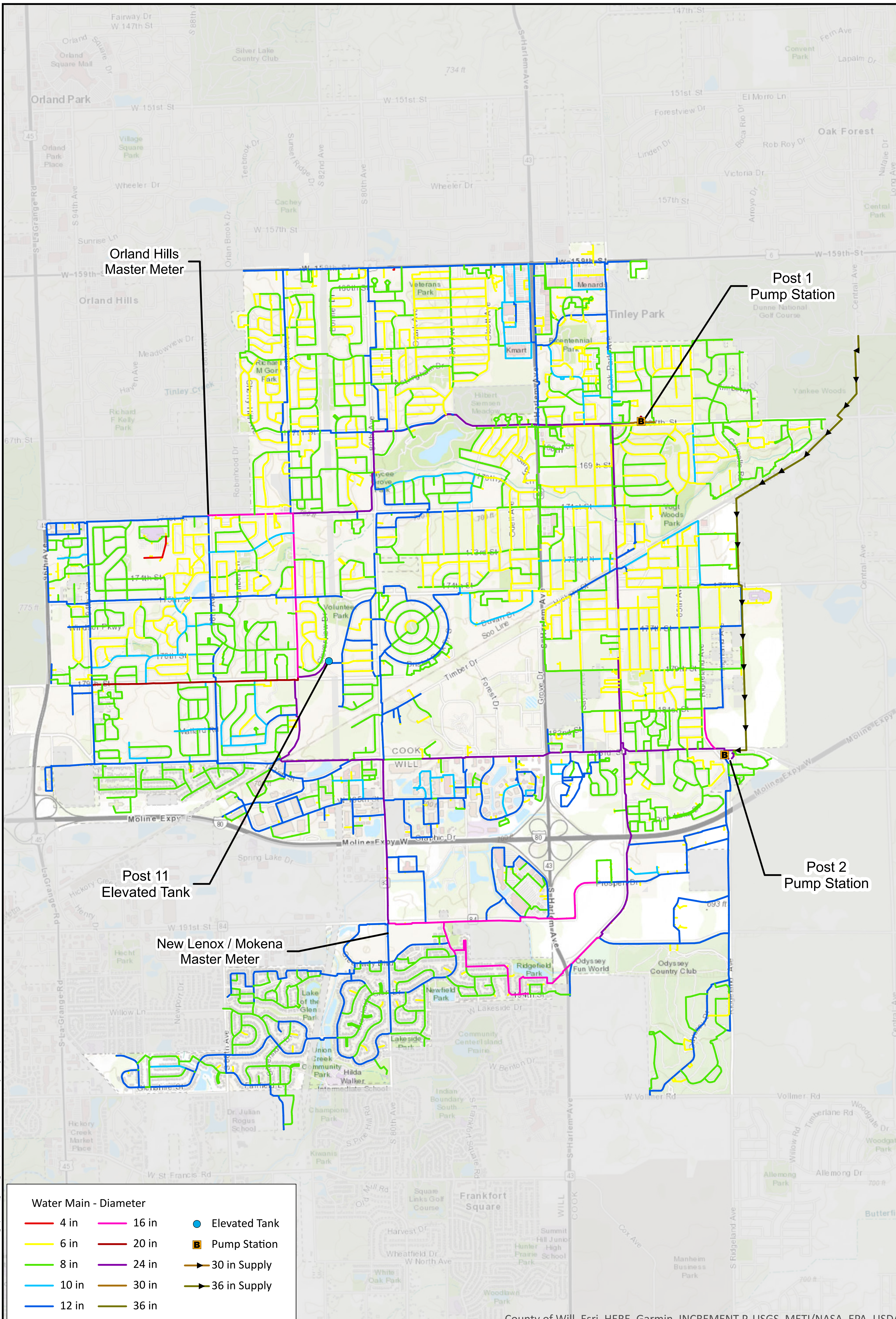
EXHIBITS

BAXTER & WOODMAN
Consulting Engineers

EXISTING WATER SYSTEM

Water Distribution System

VILLAGE OF TINLEY PARK, ILLINOIS



Source(s):Water Data received Aug 2019 - B&W 2020 Water Modeling

Exhibit A

N

Not to Scale

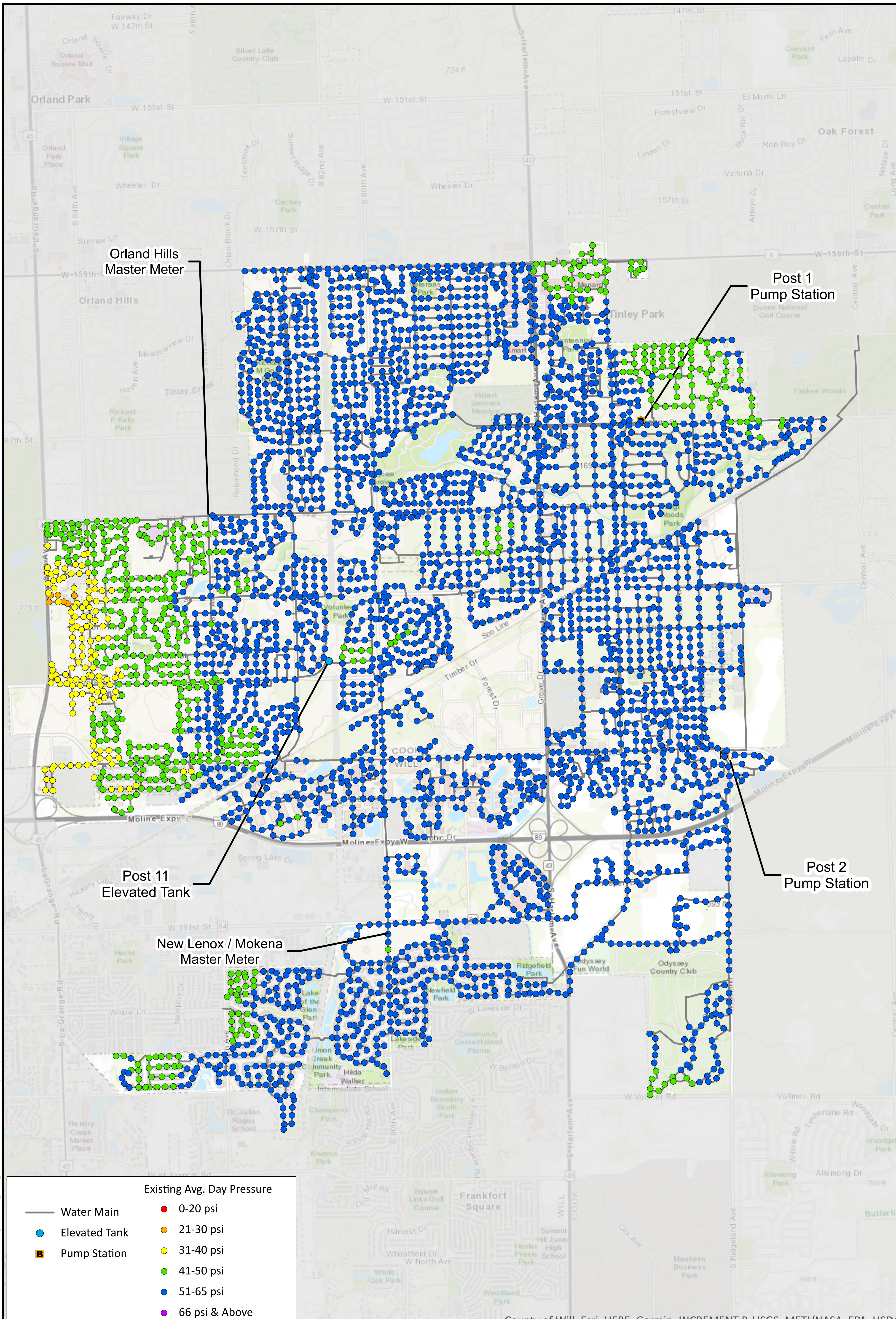
BAXTER & WOODMAN
Consulting Engineers

EXISTING SYSTEM PRESSURES

AVERAGE DAY DEMAND

Water Distribution System

VILLAGE OF TINLEY PARK, ILLINOIS



Source(s):Water Data received Aug 2019 - B&W 2020 Water Modeling

Exhibit B

N

Not to Scale

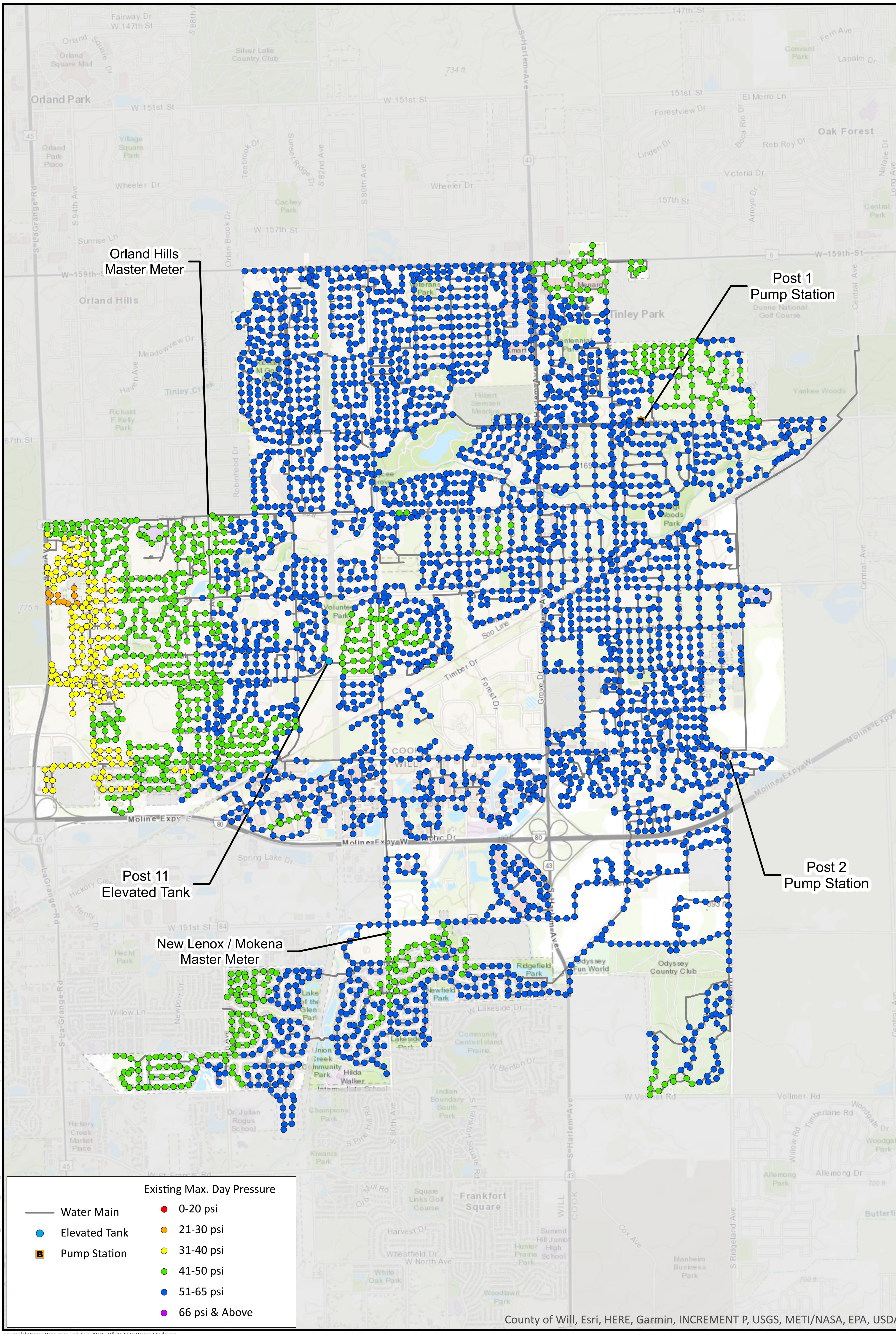
BAXTER & WOODMAN Consulting Engineers

EXISTING SYSTEM PRESSURES

MAXIMUM DAY DEMAND

Water Distribution System

VILLAGE OF TINLEY PARK, ILLINOIS



Source(s):Water Data received Aug 2019 - B&W 2020 Water Modeling

Exhibit C

N

Not to Scale

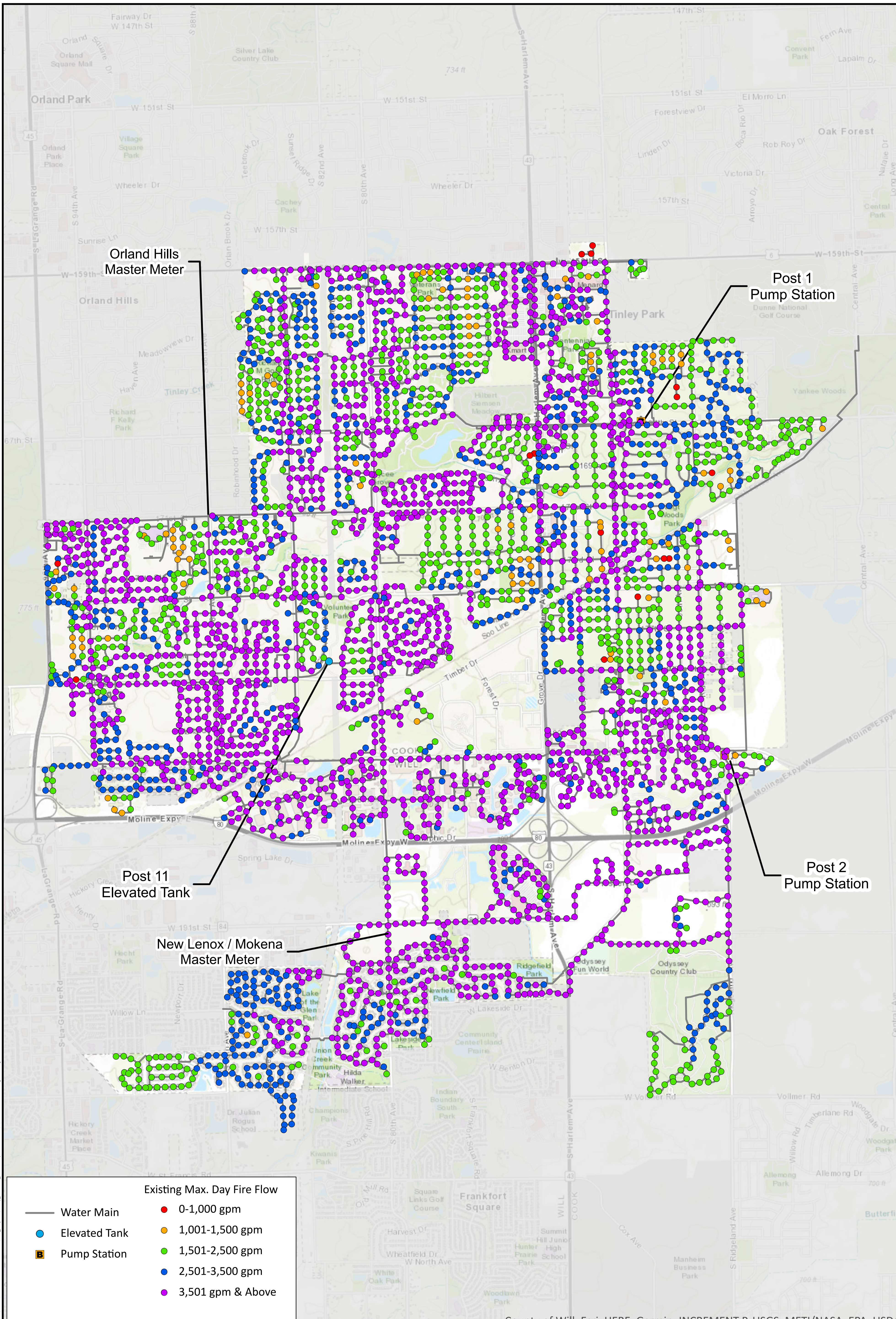
BAXTER & WOODMAN
Consulting Engineers

EXISTING SYSTEM FIRE FLOWS

MAXIMUM DAY DEMAND

Water Distribution System

VILLAGE OF TINLEY PARK, ILLINOIS



Source(s): Water Data received Aug 2019 - B&W 2020 Water Modeling

Exhibit D

Not to Scale

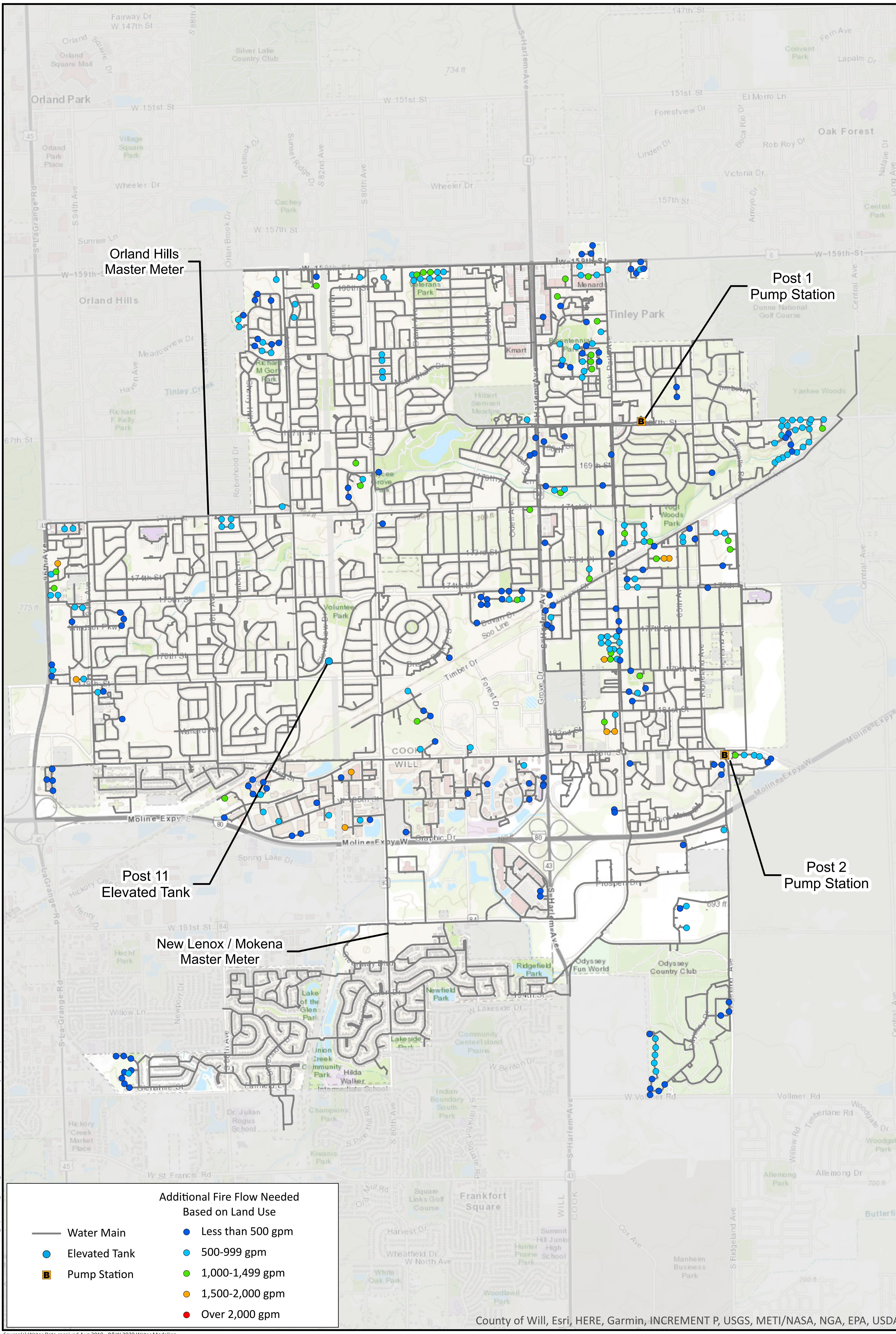
BAXTER & WOODMAN Consulting Engineers

EXISTING SYSTEM ADDITIONAL FIRE FLOW NEEDED

MAXIMUM DAY DEMAND

Water Distribution System

VILLAGE OF TINLEY PARK, ILLINOIS



Source(s):Water Data received Aug 2019 - B&W 2020 Water Modeling

Exhibit E

N

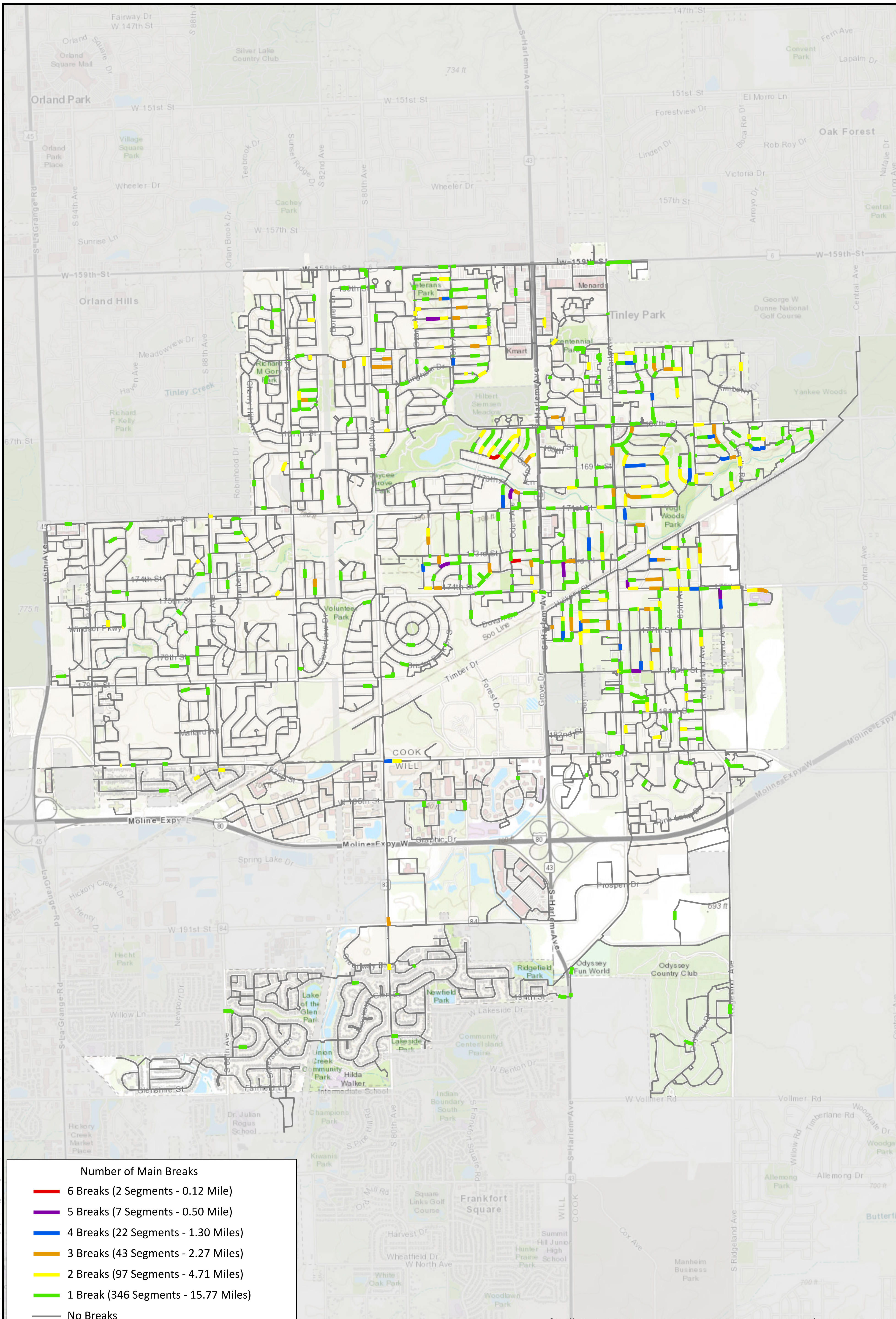
Not to Scale

BAXTER & WOODMAN Consulting Engineers

MAIN BREAK PER SEGMENT

Water Distribution System

VILLAGE OF TINLEY PARK, ILLINOIS



Source(s):Water Data received Aug 2019 - B&W 2020 Water Modeling

N

Not to Scale

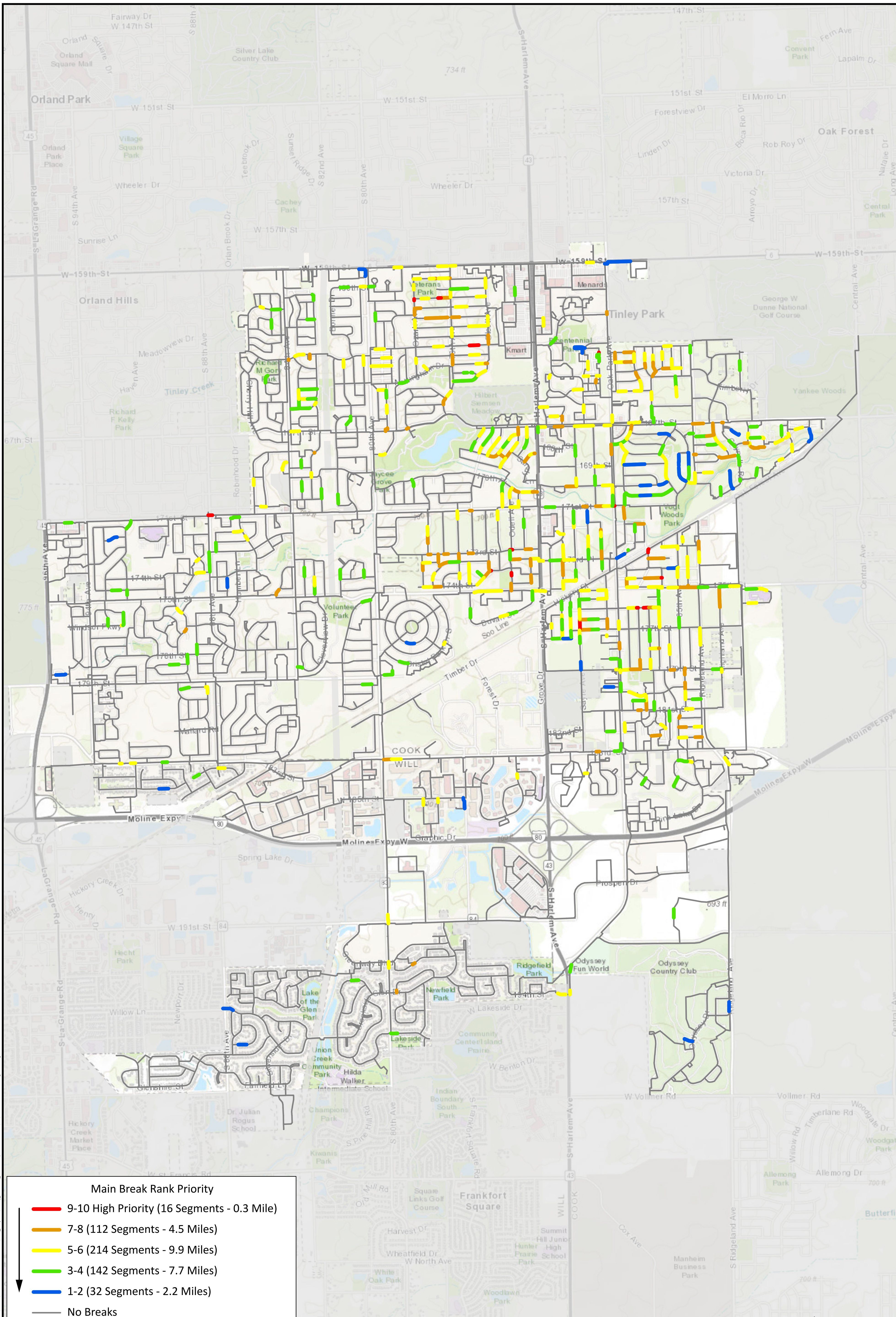
Exhibit F

BAXTER & WOODMAN Consulting Engineers

MAIN BREAK RANKING

Water Distribution System

VILLAGE OF TINLEY PARK, ILLINOIS



Source(s):Water Data received Aug 2019 - B&W 2020 Water Modeling

Exhibit G

N

Not to Scale

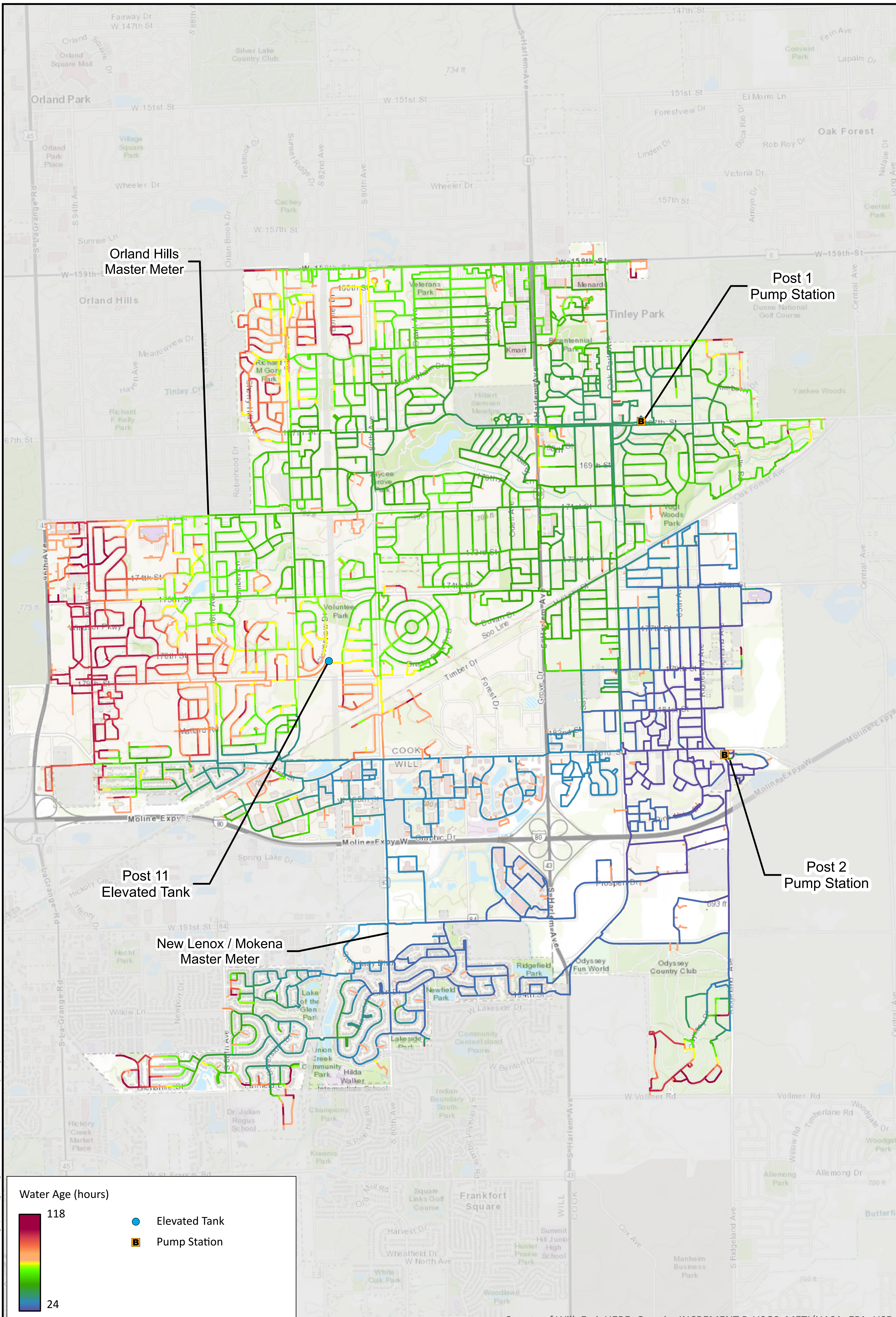
BAXTER & WOODMAN
Consulting Engineers

ESTIMATED WATER AGE

AVERAGE DAY DEMAND

Water Distribution System

VILLAGE OF TINLEY PARK, ILLINOIS



Source(s):Water Data received Aug 2019 - B&W 2020 Water Modeling

N

Not to Scale

Exhibit H

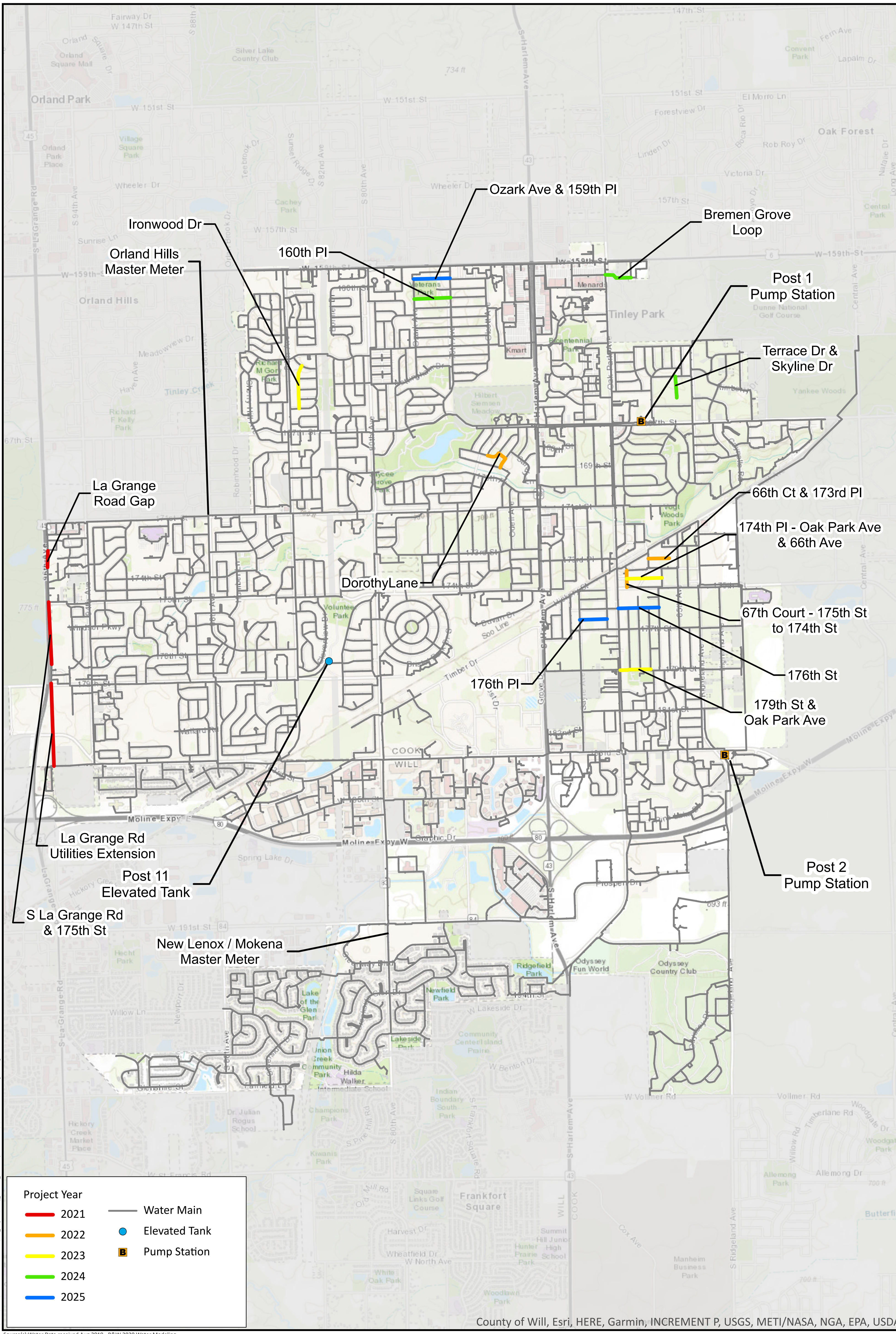
BAXTER & WOODMAN Consulting Engineers

RECOMMENDED WATER MAIN IMPROVEMENTS (2021-2025)

Water Distribution System

PROJECT LOCATIONS

VILLAGE OF TINLEY PARK, ILLINOIS



Source(s):Water Data received Aug 2019 - B&W 2020 Water Modeling

Exhibit I

N

Not to Scale

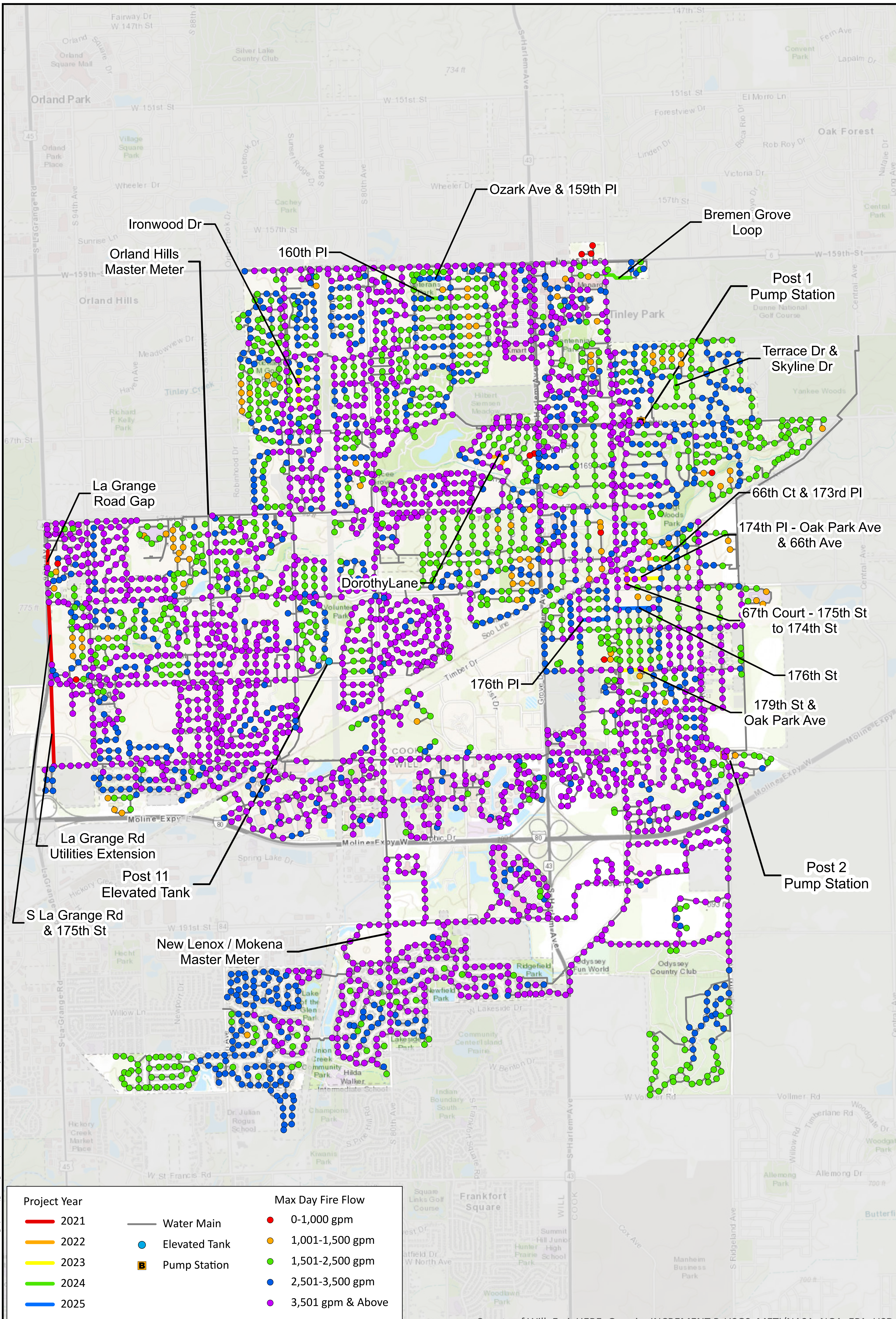
BAXTER & WOODMAN Consulting Engineers

RECOMMENDED WATER MAIN IMPROVEMENTS (2021-2025)

Water Distribution System

MAXIMUM DAY FIRE FLOWS

VILLAGE OF TINLEY PARK, ILLINOIS



Source(s):Water Data received Aug 2019 - B&W 2020 Water Modeling

Exhibit J

Not to Scale

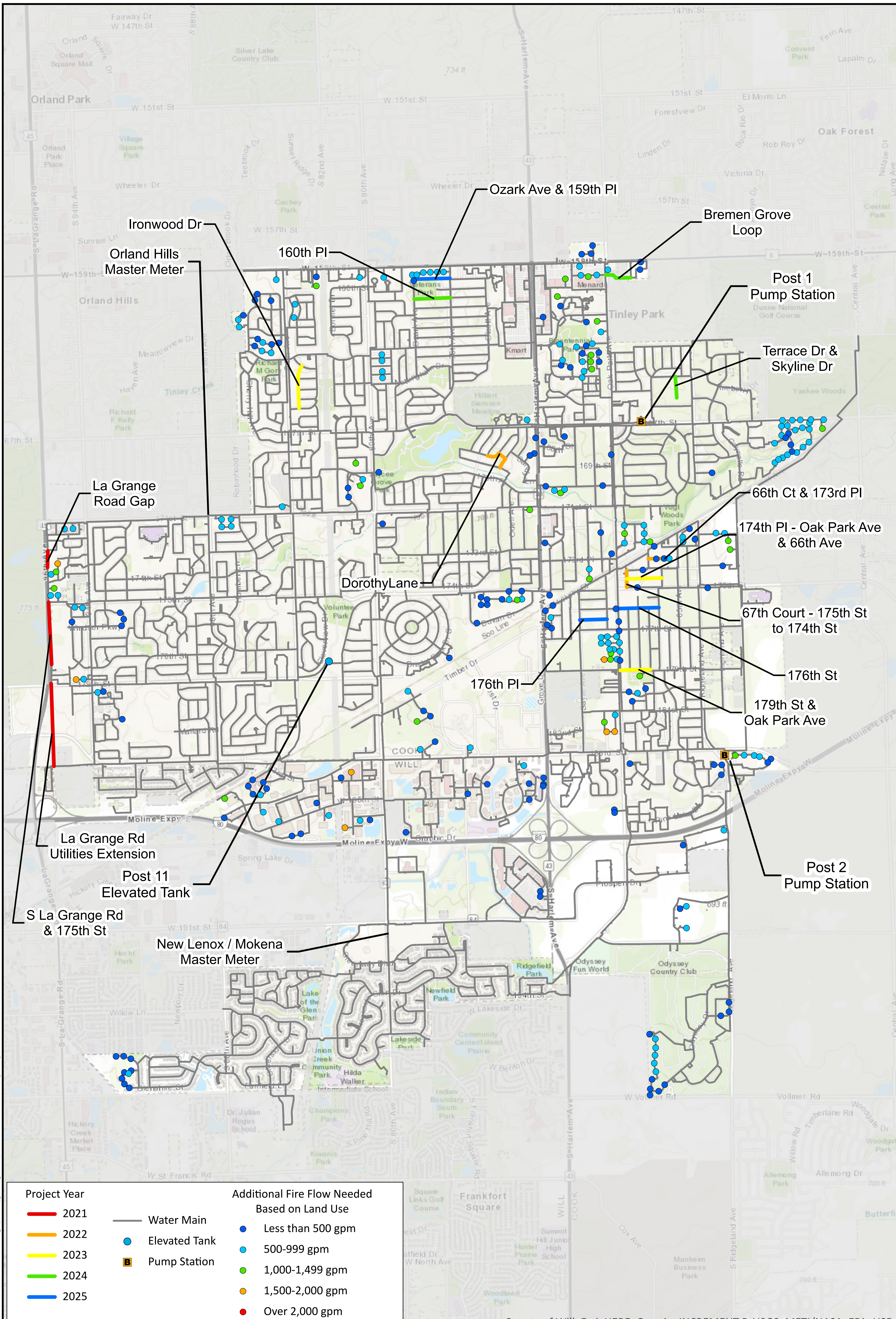
BAXTER & WOODMAN Consulting Engineers

RECOMMENDED WATER MAIN IMPROVEMENTS (2021-2025)

ADDITIONAL FIRE FLOW NEEDED MAXIMUM DAY

Water Distribution System

VILLAGE OF TINLEY PARK, ILLINOIS



Source(s):Water Data received Aug 2019 - B&W 2020 Water Modeling

N

Not to Scale

Exhibit K

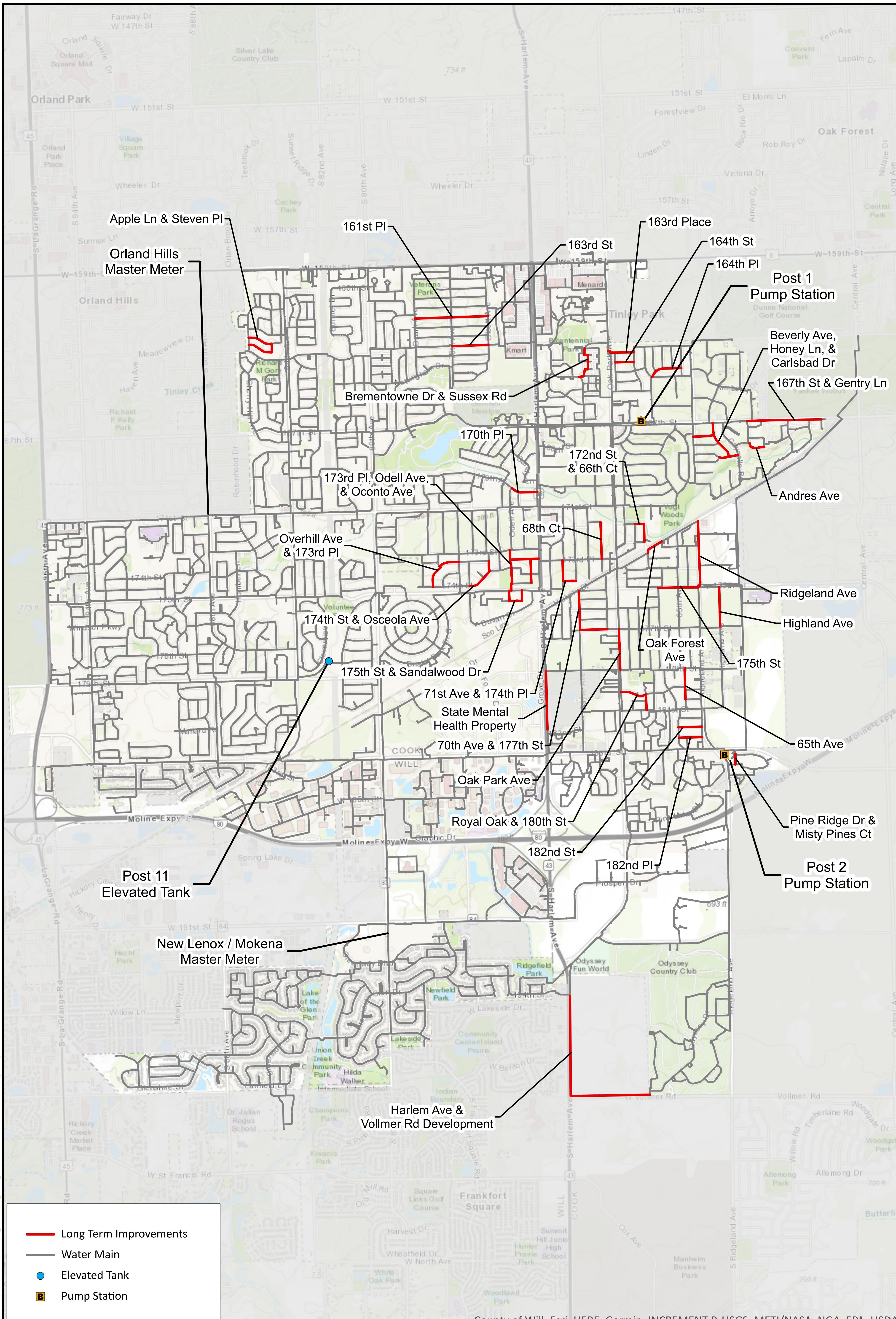
BAXTER & WOODMAN Consulting Engineers

RECOMMENDED WATER MAIN IMPROVEMENTS (LONG TERM)

Water Distribution System

PROJECT LOCATIONS

VILLAGE OF TINLEY PARK, ILLINOIS



Source(s):Water Data received Aug 2019 - B&W 2020 Water Modeling

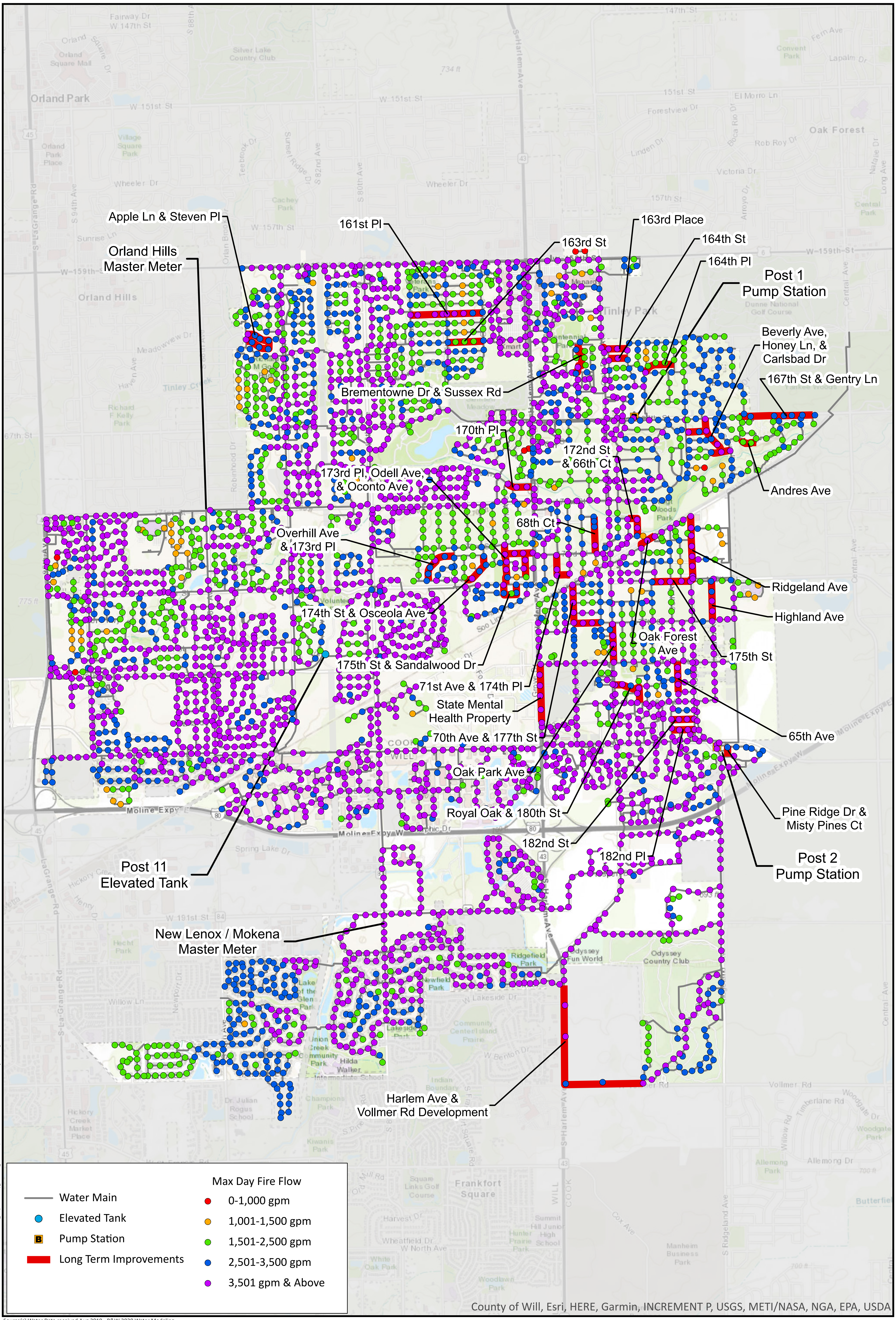
Exhibit L

N

Not to Scale

BAXTER & WOODMAN Consulting Engineers

MAXIMUM DAY FIRE FLOWS



Source(s):Water Data received Aug 2019 - B&W 2020 Water Modeling

N

Not to Scale

Exhibit M

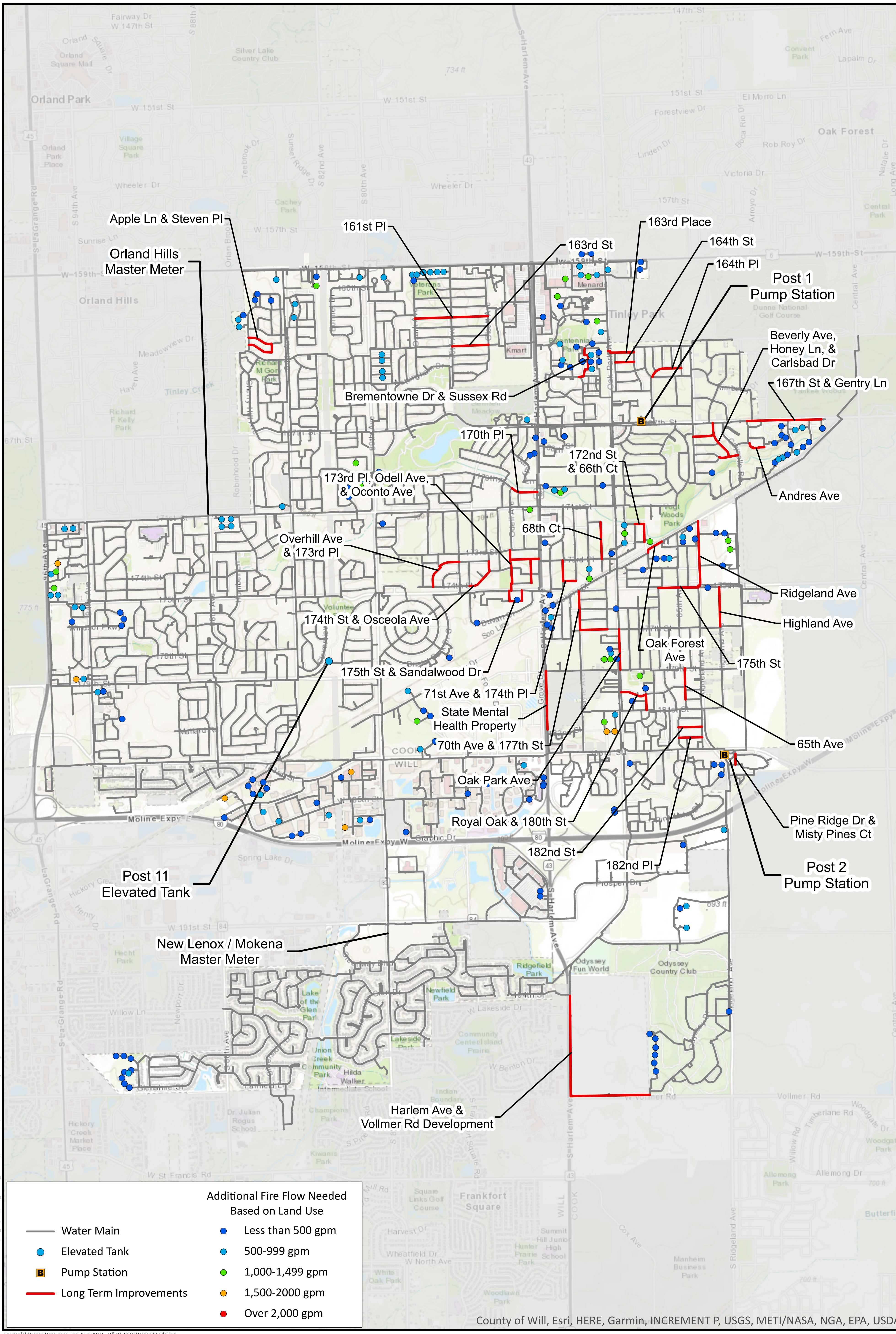
BAXTER & WOODMAN Consulting Engineers

RECOMMENDED WATER MAIN IMPROVEMENTS (LONG TERM)

Water Distribution System

ADDITIONAL FIRE FLOW NEEDED MAXIMUM DAY

VILLAGE OF TINLEY PARK, ILLINOIS



Source(s):Water Data received Aug 2019 - B&W 2020 Water Modeling

Exhibit N

N

Not to Scale

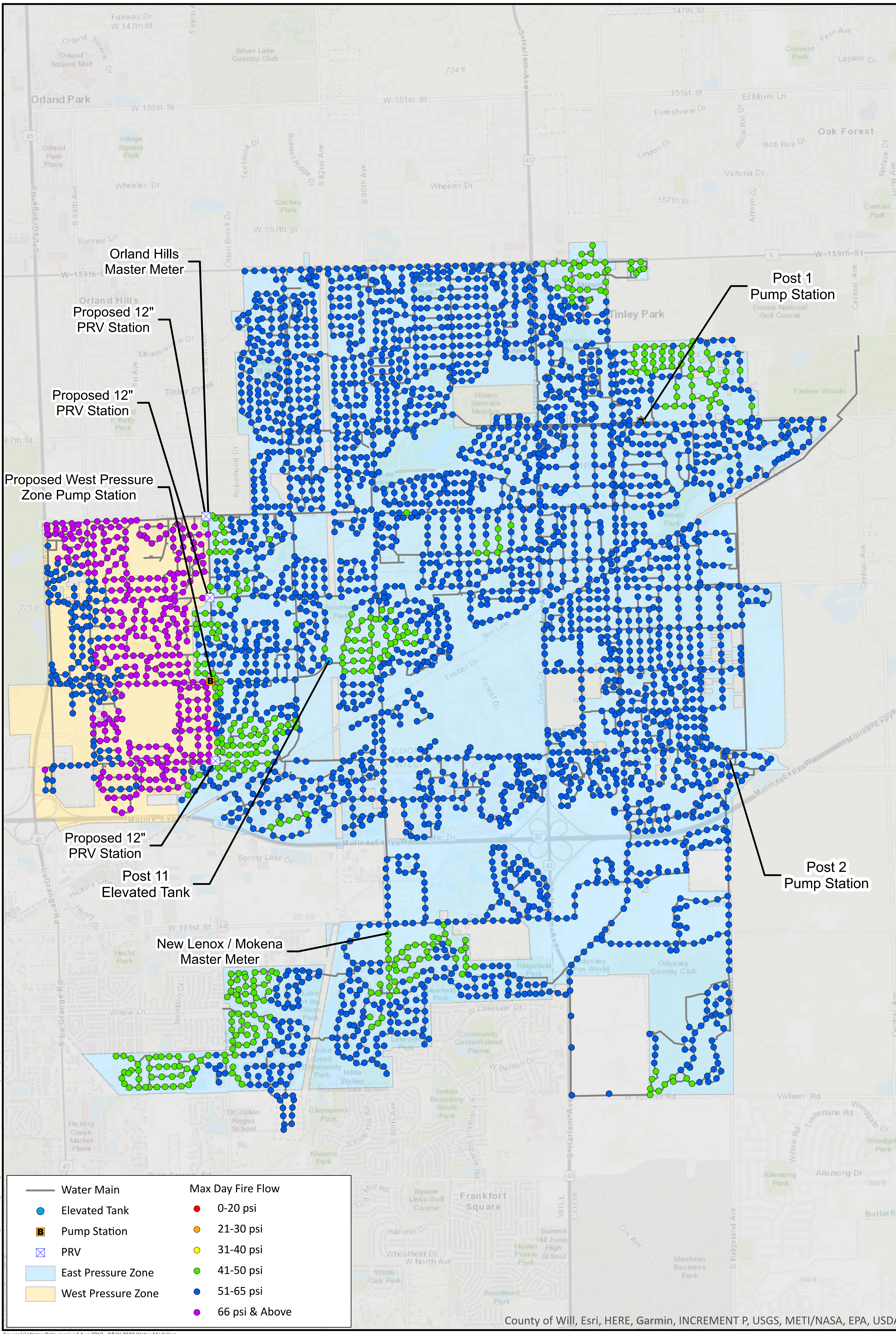
BAXTER & WOODMAN Consulting Engineers

PROPOSED WEST PRESSURE ZONE IMPROVEMENTS

Water Distribution System

MAXIMUM DAY PRESSURES

VILLAGE OF TINLEY PARK, ILLINOIS



Source(s):Water Data received Aug 2019 - B&W 2020 Water Modeling

Exhibit O

N

Not to Scale

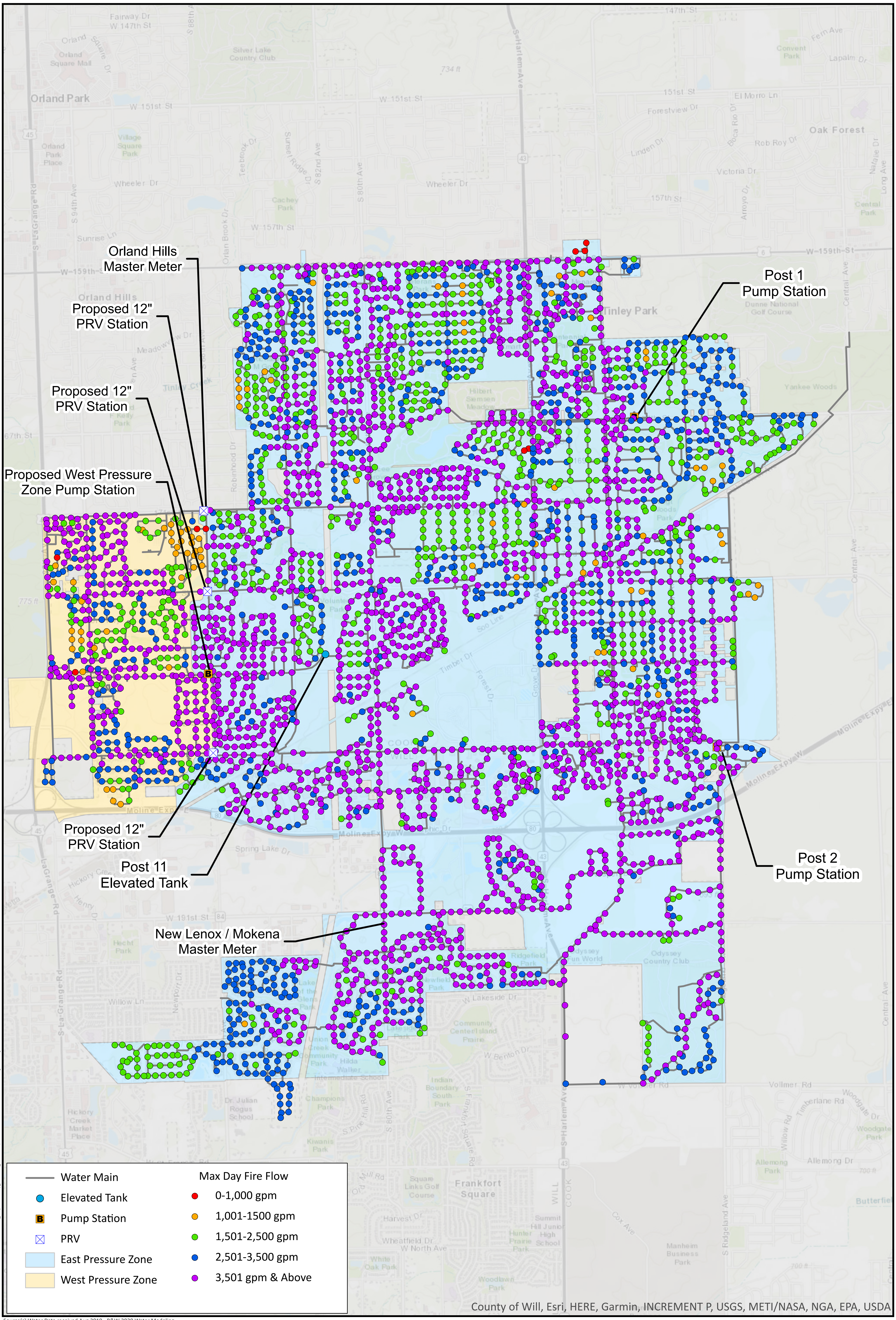
BAXTER & WOODMAN Consulting Engineers

PROPOSED WEST PRESSURE ZONE IMPROVEMENTS

Water Distribution System

MAXIMUM DAY FIRE FLOWS

VILLAGE OF TINLEY PARK, ILLINOIS



Source(s): Water Data received Aug 2019 - B&W 2020 Water Modeling

Exhibit P

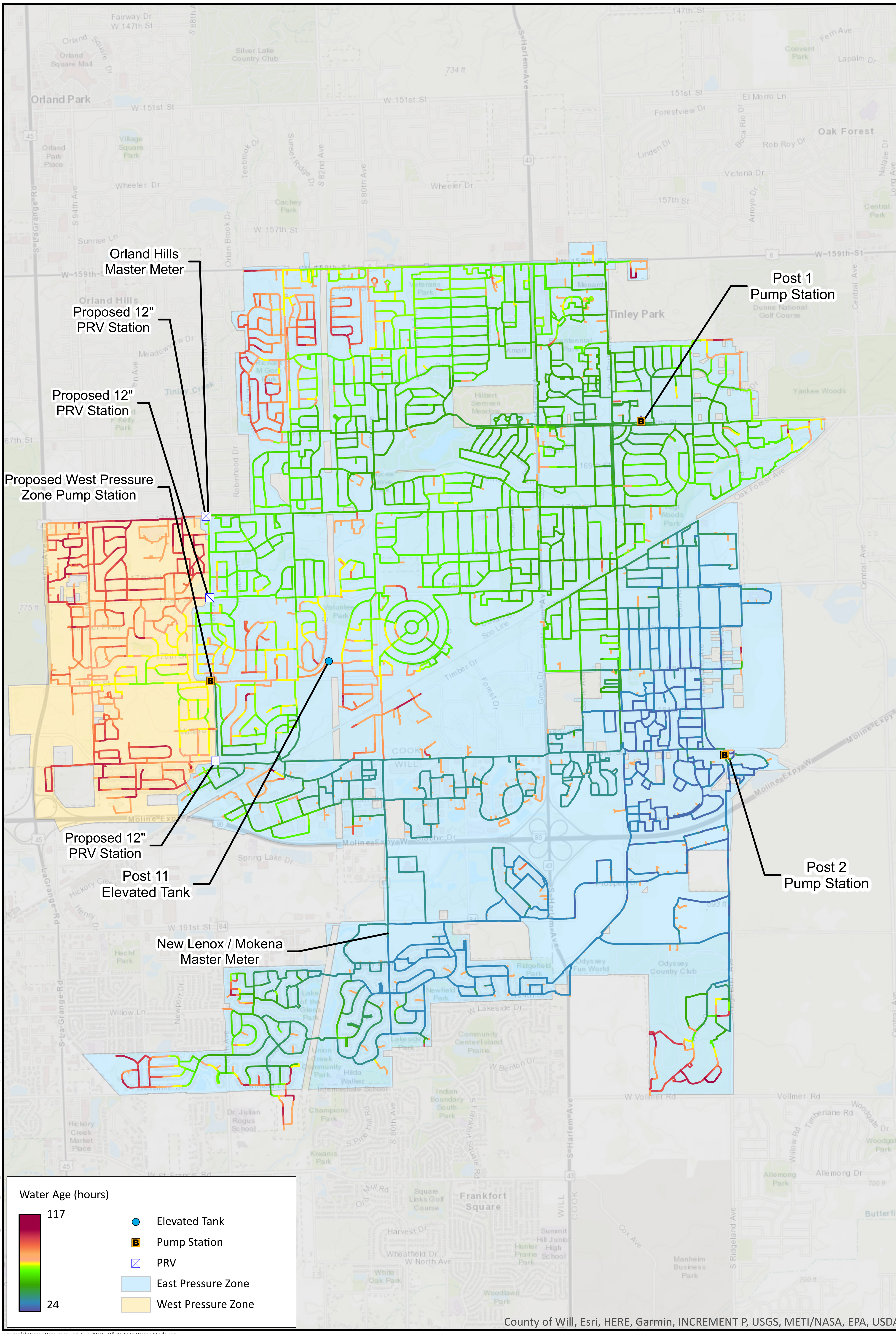
N

Not to Scale

BAXTER & WOODMAN Consulting Engineers

ESTIMATED WATER AGE WITH PROPOSED WEST PRESSURE ZONE
AVERAGE DAY DEMAND

Water Distribution System
VILLAGE OF TINLEY PARK, ILLINOIS



Source(s):Water Data received Aug 2019 - B&W 2020 Water Modeling

Exhibit Q

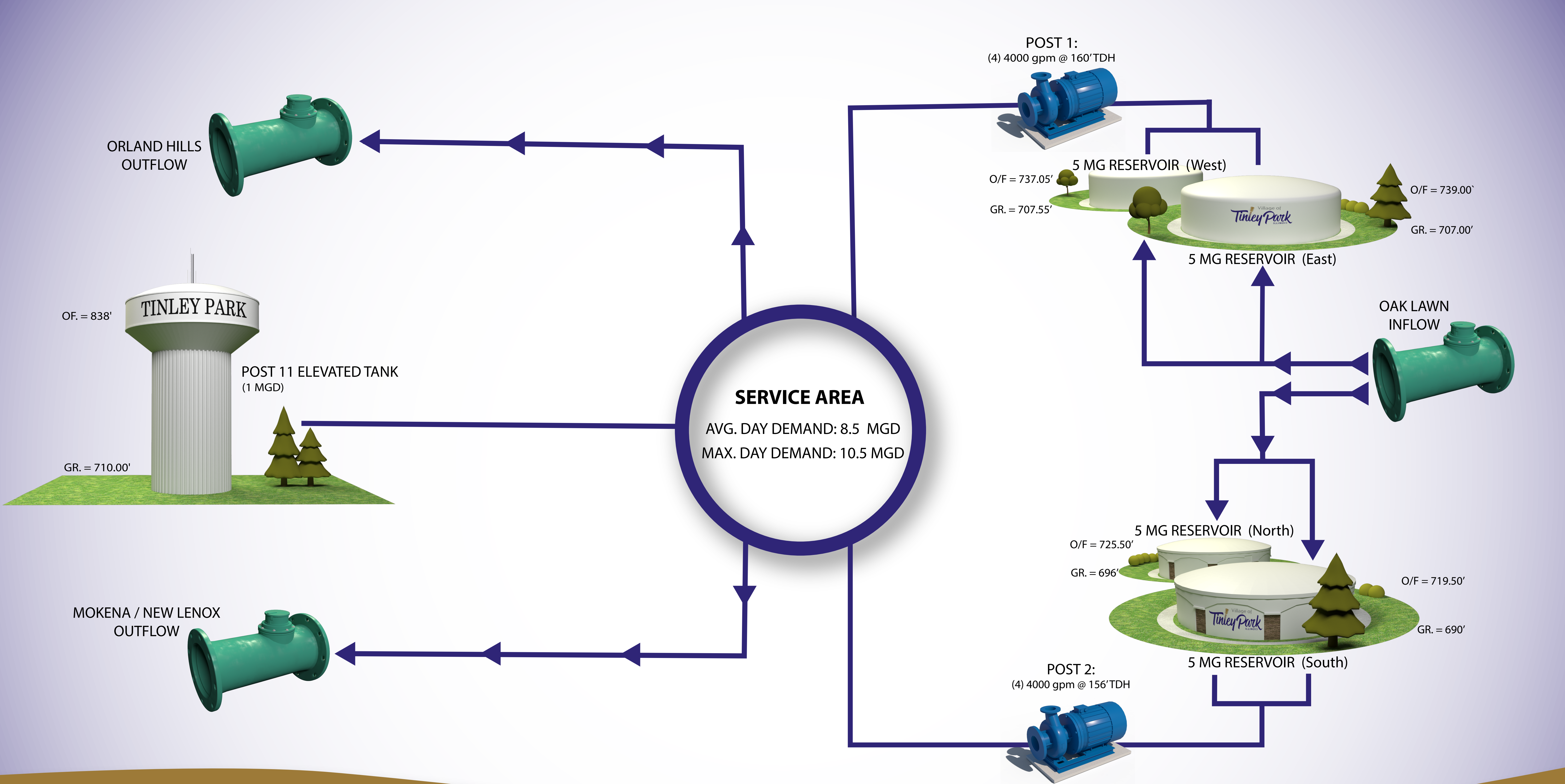
N

Not to Scale

BAXTER & WOODMAN Consulting Engineers

APPENDICES

Appendix A

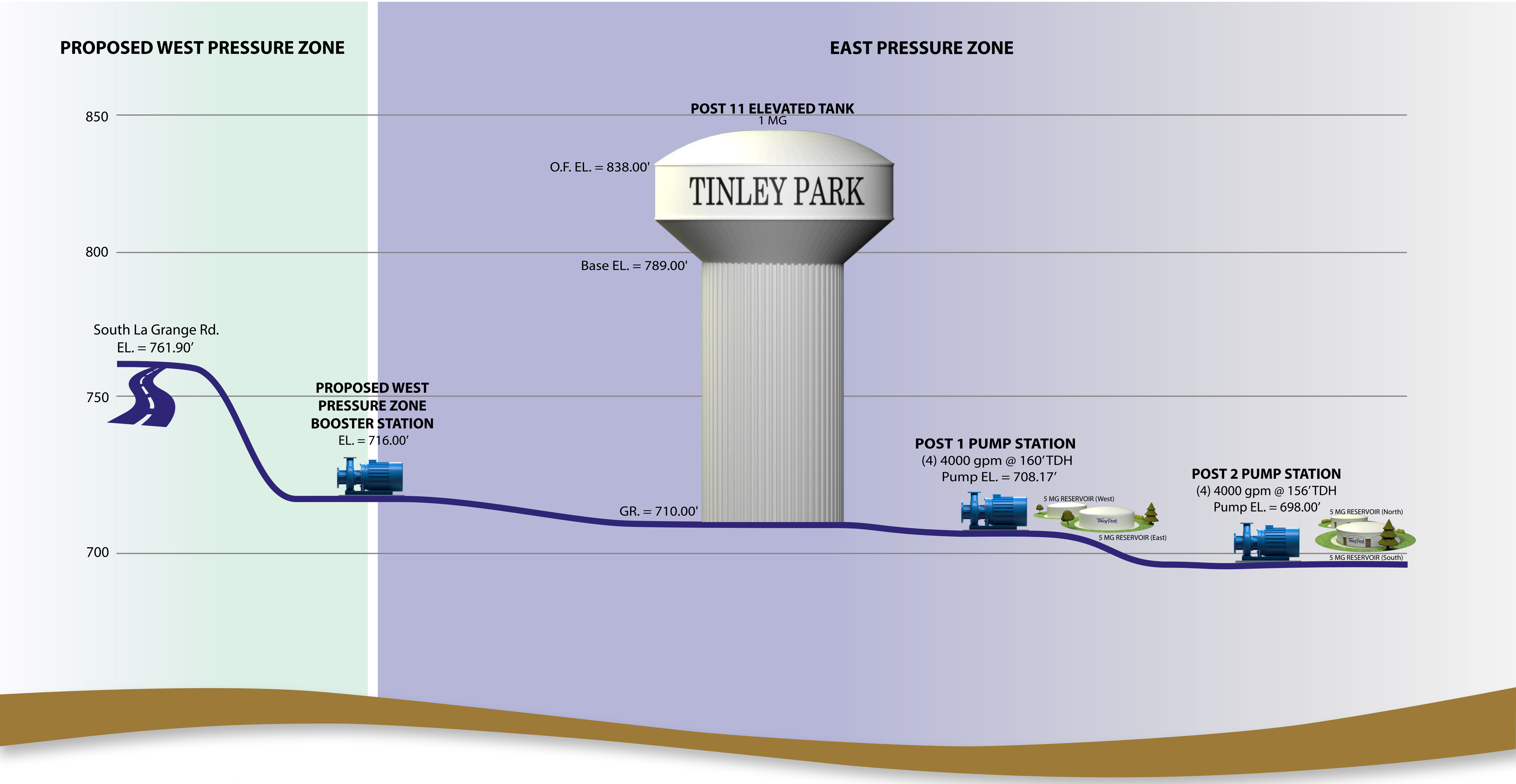


Village of Tinley Park ILLINOIS

TINLEY PARK WATER SYSTEM

BAXTER & WOODMAN Consulting Engineers

Appendix B



Village of Tinley Park ILLINOIS

PROPOSED WATER SYSTEM HYDRAULIC PROFILE

BAXTER & WOODMAN Consulting Engineers

Appendix C

Village of Tinley Park
Proposed 5-Year Capital Improvement Program (CIP)

BAXTER & WOODMAN
Consulting Engineers

Year	Final Project Names	Description	Low Fire Flow	Connectivity-Looping	Main Breaks	Main Break Ranking	Under-Sized Mains	Future Development	Critical Care User	Critical Care Definition	Installation Decade	Existing Pipe Material	Existing Diameter (in)	Proposed Diameter (in)	Length	Unit Price	Estimated Construction Cost	Estimated Design Engineering	Estimated Construction Engineering	Estimated Capital Cost
2021	La Grange Rd Utilities Extension	12" extension on S La Grange Rd from 183rd St & 179th St	X	X				X	X	Institutional	-	-	-	12	2,700	-	$ 920,000	$ 63,750	$ 79,750	$ 1,060,000
	Proposed West Pressure Zone	Above grade, (2) 750 gpm pumps @ 60' TDH w/ VFDs, (3) 12" PRV Stations						X			-	-	-	-	-	-	$ 1,680,000	$ 185,000	$ 185,000	$ 2,070,000
	S La Grange Rd & 175th St	12" extension on S La Grange from 179th to 175th St	X	X					X	Medical	-	-	-	12	2,000	$ 375	$ 750,000	$ 56,000	$ 56,000	$ 860,000
	La Grange Road Gap	12" extension on La Grange from 171st to 175th St.	X	X						-	-	-	-	12	600	$ 375	$ 225,000	$ 17,000	$ 17,000	$ 260,000
																2021 Subtotal:	**$ 3,575,000**	**$ 321,750**	**$ 337,750**	**$ 4,250,000**
2022	67th Court - 175th St to 174th St	12" extension on 67th Court from 175th St to 174th St to 66th Ave	X		X	7	X				1970	DI	6	12	1,100	$ 375	$ 410,000	$ 31,000	$ 31,000	$ 470,000
	66th Ct & 173rd Pl	Replace 6" with 8" watermain on 173rd Pl	X		X	7					1970	DI	6	8	700	$ 325	$ 230,000	$ 17,000	$ 17,000	$ 260,000
	Dorothy Ln	Replace 6" with 8" on Dorothy Ln, near Bannes School			X	8	X			Schools	1960	DI	6	8	990	$ 325	$ 390,000	$ 29,000	$ 29,000	$ 450,000
																2022 Subtotal:	**$ 1,030,000**	**$ 77,000**	**$ 77,000**	**$ 1,180,000**
2023	179th St & Oak Park Ave	179th St from Oak Park Ave to 66th Ct	X		X	8	X		X	Schools	1970	DI	6	8	1,000	$ 325	$ 330,000	$ 25,000	$ 25,000	$ 380,000
	174th Pl - Oak Park Ave & 66th Ave	8" replacement on 174th Pl from Oak Park Ave & 66th Ave	X		X	7					1960	DI	6	8	1,100	$ 325	$ 360,000	$ 27,000	$ 27,000	$ 410,000
	Ironwood Dr	Tanbark Dr to Woodland Dr			X	5					1970	DI	6	8	1,400	$ 325	$ 460,000	$ 35,000	$ 35,000	$ 530,000
																2023 Subtotal:	**$ 1,150,000**	**$ 87,000**	**$ 87,000**	**$ 1,320,000**
2024	Terrace Dr & Skyline Dr	Replace 6" with 8" watermain on Skyline Dr. (create loop)	X	X	X	6					1960	DI	6	8	1,400	$ 325	$ 460,000	$ 35,000	$ 35,000	$ 530,000
	160th Pl	160th Pl from Ozark Ave to 76th Ave			X	9					1960	DI	6	8	1,200	$ 325	$ 390,000	$ 29,000	$ 29,000	$ 450,000
	Bremen Grove Loop	New 12" loop through Bremen Grove	X	X					X	Medical	-	-	-	12	900	$ 375	$ 340,000	$ 26,000	$ 26,000	$ 390,000
																2024 Subtotal:	**$ 1,190,000**	**$ 90,000**	**$ 90,000**	**$ 1,370,000**
2025	176th St	176th St from Oak Park Ave to 66th Ave			X	9					1960	DI	6	8	1,300	$ 325	$ 420,000	$ 32,000	$ 32,000	$ 480,000
	Ozark Ave & 159th Pl	8" extension on 159th St from 76th Ave to Ozark Ave	X				X				1970	DI	6	8	1,200	$ 325	$ 390,000	$ 29,000	$ 29,000	$ 450,000
	176th Pl	176th Pl from 70th Ave to 68th Ct			X	7					1960	DI	6	8	900	$ 325	$ 290,000	$ 22,000	$ 22,000	$ 330,000
																2025 Subtotal:	**$ 1,100,000**	**$ 83,000**	**$ 83,000**	**$ 1,260,000**
Long Term	172nd St & 66th Ct	12" extension on 66th Ct	X				X				1960	DI	6	12	900	$ 375	$ 340,000	$ 26,000	$ 26,000	$ 390,000
	Beverly Ave, Honey Ln, & Carlsbad Dr	Replace 6" with 8"			X	8					1970	DI	6	8	2,500	$ 325	$ 820,000	$ 62,000	$ 62,000	$ 940,000
	Andres Ave	Andres Ave from Helen Sandidge Ct to Jennifer Ave			X	7					1980	DI	6	8	700	$ 325	$ 230,000	$ 17,000	$ 17,000	$ 260,000
	175th St & Sandalwood Dr	Upsize 6" on Sandalwood to 8"	X				X				1960	DI	6	8	1,100	$ 325	$ 360,000	$ 27,000	$ 27,000	$ 410,000
	Oak Park Ave	Upsize 6" to 8" from 179th Pl to 177th St	X				X				1970	DI	6	8	1,300	$ 325	$ 420,000	$ 32,000	$ 32,000	$ 480,000
	Pine Ridge Dr & Misty Pines Ct	Complete 8" loop on Misty Pines Dr	X	X							2000	DI	8	8	300	$ 325	$ 100,000	$ 8,000	$ 8,000	$ 120,000
	Apple Ln & Steven Pl	Upsize 6" to 8" loop on 162nd Pl, 85th Ave, & Steven Pl	X				X				1980	DI	6	8	1,900	$ 325	$ 620,000	$ 47,000	$ 47,000	$ 710,000
	Brementowne Dr & Sussex Rd	8" extension on Brementowne south to Manchester St	X								1970	DI	6	8	1,300	$ 325	$ 420,000	$ 32,000	$ 32,000	$ 480,000
	167th St & Gentry Ln	Replace 8" with 12" on 167th from Trail View Ct to Anne Marie Ave	X								1970	DI	8	12	2,500	$ 375	$ 940,000	$ 71,000	$ 71,000	$ 1,080,000
	Ridgeland Ave	Ridgeland Ave from 175th St to Oak Forest Ave	X		X	8					1970	DI	10	12	2,100	$ 375	$ 790,000	$ 59,000	$ 59,000	$ 910,000
	Oak Forest Ave	Oak Forest Ave from 66th Ct to 66th Ave			X	9	X				1960	DI	6	12	600	$ 375	$ 230,000	$ 17,000	$ 17,000	$ 260,000
	163rd St	163rd St from 76th Ave to Olcott Ave			X	9					1960	DI	6	8	1,200	$ 325	$ 390,000	$ 29,000	$ 29,000	$ 450,000
	70th Ave & 177th St	70th Ave from Hickory St to 177th St, 177th St from 70th Ave to 68th Ct			X	9					1960	DI	8	8	2,200	$ 325	$ 720,000	$ 54,000	$ 54,000	$ 830,000
	175th St	175th St from Ridgeland Ave to 66th Ave			X	9					1970	DI	8	8	1,200	$ 325	$ 390,000	$ 29,000	$ 29,000	$ 450,000
	Royal Oak & 180th St	Royal Oak from 181st St to 180th St, 180th from Royal Oak to OPA	X		X	7					1970	DI	6	8	1,300	$ 325	$ 420,000	$ 32,000	$ 32,000	$ 480,000
	163rd Place	163rd Pl from Oak Park Ave to 66th Court			X	8					1960	DI	8	8	900	$ 325	$ 290,000	$ 22,000	$ 22,000	$ 330,000
	164th St	164th St from 67th Ct to 66th Ct			X	8					1960	DI	6	8	600	$ 325	$ 200,000	$ 15,000	$ 15,000	$ 230,000
	170th Pl	170th Pl from S Harlem Ave to Odell Ave			X	8					1970	DI	6	8	900	$ 325	$ 290,000	$ 22,000	$ 22,000	$ 330,000
	173rd Pl, Odell Ave, & Oconto Ave	Replace 6" with 8"			X	9					1950	DI	6	8	3,100	$ 325	$ 1,010,000	$ 76,000	$ 76,000	$ 1,160,000
	71st Ave & 174th Pl	71st Ave from 173rd Pl to 70th Ave			X	8					1960	DI	6	8	1,000	$ 325	$ 330,000	$ 25,000	$ 25,000	$ 380,000
	Highland Ave	Highland Ave from 177th St to 175th St			X	8					1970	DI	6	8	1,300	$ 325	$ 420,000	$ 32,000	$ 32,000	$ 480,000
	161st Pl	161st Pl from Ozark Ave to Olcott Ave			X	7					1960	DI	6	8	2,400	$ 325	$ 780,000	$ 59,000	$ 59,000	$ 900,000
	164th Pl	164th St from Terrace Dr to 64th Ct			X	7					1960	DI	6	8	1,000	$ 325	$ 330,000	$ 25,000	$ 25,000	$ 380,000
	174th St & Osceola Ave	174th St from Oleander Ave to 173rd Pl			X	7					1960	DI	6	8	1,200	$ 325	$ 390,000	$ 29,000	$ 29,000	$ 450,000
	Overhill Ave & 173rd Pl	Overhill Ave from 174th St to Oriole Ave			X	7					1960	DI	6	8	1,400	$ 325	$ 460,000	$ 35,000	$ 35,000	$ 530,000
	68th Ct	68th Ct from 173rd Pl to 172nd St			X	7					1960	DI	6	8	1,200	$ 325	$ 390,000	$ 29,000	$ 29,000	$ 450,000
	65th Ave	65th Ave from 180th Pl to 179th St			X	7					1970	DI	6	8	1,000	$ 325	$ 330,000	$ 25,000	$ 25,000	$ 380,000
	182nd St	182nd St from 66th Ave to Ridgeland Ave			X	7					1970	DI	6	8	700	$ 325	$ 230,000	$ 17,000	$ 17,000	$ 260,000
	182nd Pl	182nd Pl from 66th Ave to Ridgeland Ave			X	7					1970	DI	6	8	700	$ 325	$ 230,000	$ 17,000	$ 17,000	$ 260,000
	Harlem Ave & Vollmer Rd Development	12" connection on Harlem Ave & Vollmer Rd						X			-	-	-	12	5,800	$ 375	$ 2,200,000	$ 165,000	$ 165,000	$ 2,530,000
	State Mental Health Property	12" Connection on Harlem from 183rd St to 179th St						X			-	-	-	12	1,900	$ 375	$ 700,000	$ 53,000	$ 53,000	$ 810,000
																Long Term Subtotal:	**$ 15,770,000**	**$ 1,188,000**	**$ 1,188,000**	**$ 18,110,000**
																Total:	**$ 23,800,000**	**$ 1,800,000**	**$ 1,900,000**	**$ 27,500,000**

1. Prices include water service replacement/adjustment, new valves, new hydrants, trench backfill, pavement or lawn restoration, traffic control, erosion control, construction layout, and mobilization. Water main replacement costs also assumes street reconstruction is being completed.
2. Prices do not include right-of-way acquisition, temporary or permanent easements, or relocating other utilities.
3. Prices are current for 2020.