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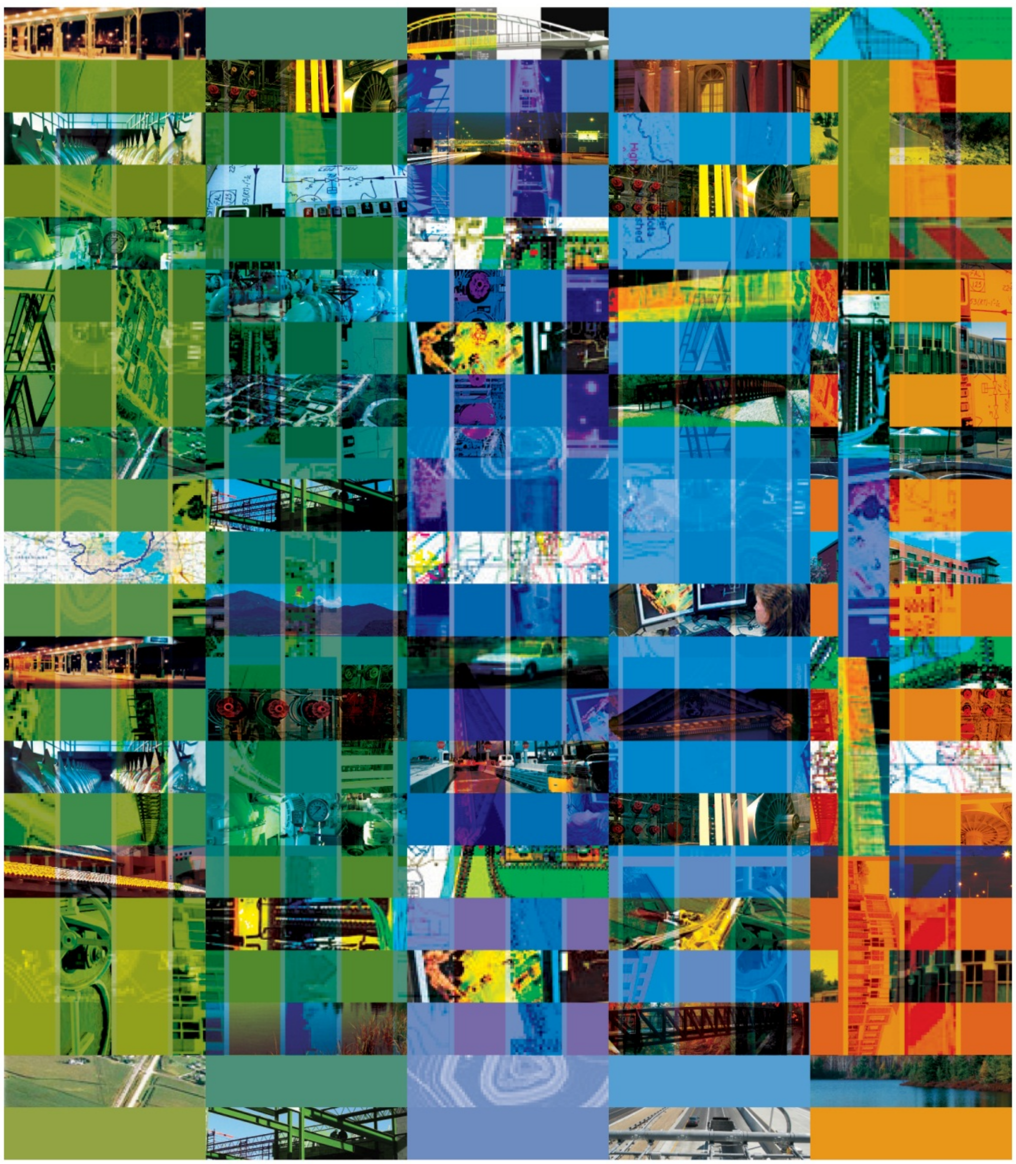
Water Distribution System Plan

Report

Village of

Glencoe, IL

February 2016



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Report for

Village of Glencoe, Illinois

Water Distribution System Plan

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February 2016

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EXECUTIVE SUMMARY

PURPOSE AND SCOPE OF STUDY

The Village of Glencoe (Village) operates a conventional water treatment plant [6 million gallons per day (mgd)] and distribution system to serve its 8,723 customers (2010 United States Census data). The treated water is pumped into a distribution system consisting of 58 miles of water main of various sizes between 4 inches to 16 inches, a 0.5 million gallon (MG) elevated tank located on Tower Road, and a 2 MG ground level reservoir located near the water treatment plant. The Village commissioned this study to evaluate how the existing water system is performing, evaluate water distribution system improvements that address water system deficiencies based on system pressures, fire flow availability, and water main break data, and improve overall system operation and redundancy. The results of these evaluations have been compiled toward water system improvement recommendations and a 20-year capital improvement program that supplements the *Water Supply Planning Report* completed in March 2015.

Several analyses were performed throughout the course of this study. The first analysis evaluated water system operations under a peak demand scenario to identify areas within the water system that are below minimum levels of service that the Village has established for its operations. To improve system pressures to a level of service at or greater than 40 pounds per square inch (psi) under all scenarios, it is recommended to construct a new elevated tank with an operating range higher than the existing tank in area of Elder Court and Forestway Drive as described in Section 2 of this report. To improve system fire flows to a minimum 500 gpm under all scenarios, additional water main reinforcement and replacement is recommended within the Village's prioritized 20-year capital improvements program in Section 3 of this report. The recommendations for water main improvements to increase fire flow conditions within this report include six projects which are prioritized in the first few years of the 20-year capital improvement program.

The second analysis evaluated water main break data collected between 2000 and 2015 within the Village. The water main break data helped to identify sections of water main that have experienced higher than average failure compared to other sections of water main because of a variety of factors identified in Section 2 of this report. Replacement of the existing water main was prioritized based on the number of breaks per 100 feet of water main, promoting replacement of pipes with a high number of breaks sooner compared to pipes with fewer number of breaks later within the 20 -year planning period. This analysis was also coordinated with previous water system improvement plans and upcoming roadway maintenance plans being considered by the Village. The recommendations for water main replacement within this report include 40 projects, prioritized and evenly distributed over the next 20 years, to help maintain and/or reduce the number of water main failures within the Village.

The recommended prioritized water main replacement plan represents a total present worth probable cost of $13,991,000, or approximately $700,000 per year average investment to maintain the water distribution system within the Village. Fire flow related improvement projects identified in our first analysis have an opinion of probable cost of $818,000. Water main replacement improvement projects identified in our second analysis have an opinion of probable cost of $13,172,000.

A third analysis addressed the feasibility of installing a new water main to reinforce the single transmission main connecting the Village's water distribution system and the existing 0.5 MG elevated tank. The existing 10-inch and 14-inch water main under the Skokie Lagoons and across the Eden's Expressway represents critical system infrastructure, that if not reinforced, could represent a significant challenge to the operation of the Village's water system. The recommended water reinforcement water main along Forestway Drive and Tower Road would allow the Village to continue operating with the existing elevated tank. The opinion of probable cost to install the recommended reinforcement water main is approximately $6,325,000.

A fourth analysis addresses the feasibility of constructing a new elevated water tank to replace the existing elevated tank and improve the level of service and overall system redundancy and operations. Along with the features identified in the first analysis, this analysis considered possible elevated tank locations and storage volumes that worked best within the existing distribution system and improved upon the current level of service. Of the three sites evaluated, the recommended location for a new elevated tank is in the area south of Elder Court along Forestway Drive. The Village has two options for the elevated tank size. A 0.5 MG elevated storage tank, with an opinion of probable cost of approximately $2,600,000, would match the existing tank volume, however, it would not allow the Village to take the existing 2 MG groundlevel reservoir out-of-service under all scenarios. A larger 0.75 MG elevated storage tank, with an opinion of probable cost of approximately $3,645,000, would provide the necessary storage to meet current demand and operational scenarios, including modifications and maintenance of the existing 2.0 MG reservoir.

Finally, a fifth analysis was conducted regarding the energy efficiency of the existing water system operations, mainly involving energy use by the high lift pump station at the existing WTP. The existing pump operations are energy-efficient and use best management practices based on interviews with staff and evaluations of the available pump and water system data. A strategy to maintain optimal pump operations and energy conservation has been presented in Section 2 of this report.

In summary, the recommendations of this project are divided into a 20-year capital improvement program and individual capital projects, including water main reinforcement and elevated tank replacement. The sequencing of the proposed capital improvement program may rely on funding priorities set by the Village and are ultimately recommended to maintain the minimum level of service desired by the Village and to maintain a manageable number of water main breaks within the Village. The capital improvement projects within this Report are recommended to reduce the risk associated with distribution system failures and to aid in the transition of the water system, based on options selected out of the *2015 Water Supply Planning Report.*

SECTION 1
EXISTING DISTRIBUTION SYSTEM

1.01 INTRODUCTION

The purpose of this *Water Distribution System Plan* is to supplement the *Water Treatment Plant Master Plan* completed in March 2015 with additional information about the existing distribution system. The scope of effort associated with this plan includes the following:

1. Conduct an initial project kickoff meeting with the Village of Glencoe (Village) to collect and review existing data, including geographical information system (GIS) information, previous studies, water main break history, and the Village's roadway replacement plan.

2. Using the Village's existing calibrated water system model and available data, perform the following:

    a. Analyze the existing water distribution system under a peak water demand scenario and determine potential system deficiencies. Identify possible system improvements to address identified deficiencies.

    b. Review and analyze water main break and failure data to identify possible system improvements or operational adjustments that may reduce failure rates.

    c. Review the feasibility and cost to replace the transmission main to the Village's existing elevated water storage tank.

    d. Review the feasibility and cost to construct an additional or replacement elevated water storage tank in another location that improves system performance in the water model.

3. Prepare a technical memorandum of the findings and attend a workshop meeting with the Village to discuss the findings to date and determine the direction for next steps.

4. Using the calibrated water model, analyze and review the existing water system to determine energy requirements and potential for energy conservation and cost savings by modification of current operational procedures and system components.

5. Develop a list of short-term and long-term water system improvements. Develop opinions of probable construction cost for the recommended improvements and prioritize the improvements in a 20-year capital improvement plan.

6. Provide a report detailing the study's findings and recommendations and discuss with the Village.

1.02 EXISTING SYSTEM DEFICIENCIES

This section describes how the calibrated water model was set up and the assumptions used when modeling, identifies low service pressure areas under peak demands, identifies areas of low fire flows, and summarizes water main break data in the distribution system. Subsequent sections will identify potential water system improvements to help sustain or improve current service levels.

A. Model Conditions

The Village's calibrated water model that was created for the 2015 *Water Supply Planning Report* was used to assist in predicting the impacts of water supply changes and determine possible existing system deficiencies using steady-state analyses. Because of the complex high lift pump operation, which includes a combination of automated variable speed pumps and manually controlled full-speed pumps, it was not possible to accurately model the pump controls. The model was analyzed using a peak-hour demand condition, which is defined as the Village's maximum hour of the maximum day, and is equal to approximately 6,220 gallons per minute (gpm), or 8.95 million gallons per day (mgd). While this is not considered the peak instantaneous demand, it represents a conservative condition that may occur within the distribution system. Water was delivered from the water treatment plant (WTP) using the firm high lift capacity, which is defined as the total amount of water that can be pumped with the largest unit out of service. For each model run, High Lift Pump Nos. 1, 2, and 3 were running.

Ground level at the base of the existing 0.5 million gallons (MG) elevated tank, referred to as the Tower Road Tank, was determined to be approximately 629 feet above mean sea level (AMSL) using Village-provided digital elevation models. Construction drawings of the Tower Road Tank show a distance of 155 feet from ground level to the overflow. Therefore, the overflow elevation was calculated to be approximately 784 feet AMSL by adding the height to the ground elevation. When using the model, the water level in the Tower Road Tank was set to 10 feet below overflow, or 774 feet AMSL, to reflect a normal operating level.

Two general types of steady-state simulations were performed with the model, domestic (nonfire) and fire flow.

A steady-state simulation evaluates the operating behavior of the system at a specific point in time under steady-state (unchanging) conditions. Using this type of analysis, the behavior of pump, tank, and supply/storage relationships can be determined. It can also be used for determining pressures and flow rates within the distribution system.

A fire flow simulation provides an instantaneous snapshot of the amount of water available at points within the system while still maintaining a minimum 20 pounds per square inch (psi) residual pressure. The model simulates a separate fire event at each junction in the system and increases the flow until either the node itself or any point in the system reaches the 20 psi residual pressure threshold. Very high available fire flows (over 5,000 gpm) are typically not considered realistic but indicate areas of very strong hydraulic connectivity to sources of supply. Available fire flow will be limited by location of the hydrant relative to the model junction, diameter of the hydrant outlet, and type of firefighting equipment used.

B. Peak Hour-Domestic Only (Nonfire)

For this simulation, the WTP high lift pumps supplied 5,464 gpm (7.87 mgd) while the Tower Road Tank supplied 758 gpm (1.09 mgd), which was only 12 percent of the total supply. The model indicated that system operating pressures under peak demands were estimated to be between 33 and 88 psi, as shown by the pressure contours generated by the model in Figure 1.02-1. This range is below the minimum 35 psi normal working pressure value suggested by *Ten States Standards-Recommended Standards for Water Works*. The upper range of the system operating pressure is high, but not considered excessive, and is located adjacent to the existing WTP along the lakefront. The area of lowest pressure, in the northwest part of the Village, appears to be a result of higher ground elevations. The pressure of the Green Bay Road transducer, located in the northwest part of the Village, was estimated to be 34.3 psi. System pressures below 35 psi are indicative of low available fire flows and may lead to customer complaints of poor water quality or inadequate supply. To prevent these issues, the Village desires to maintain a minimum pressure of 40 psi. To increase the pressure in this area up to the minimum desired level of 40 psi, there are a two potential options which would include:

1. Increase the overflow elevation of the elevated tank.
2. Create a separate boosted pressure zone.

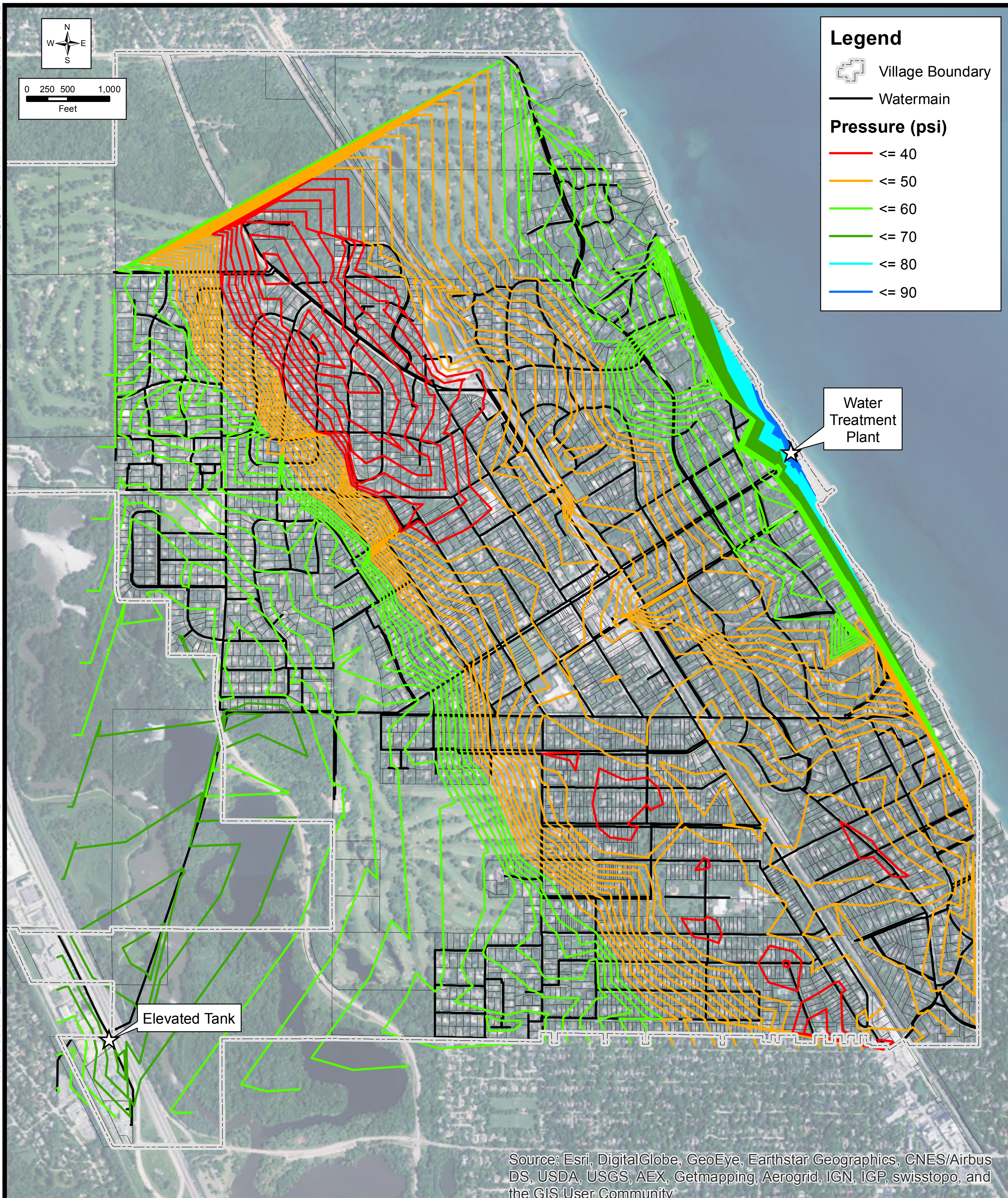
Option 1 will be discussed further in Section 2 of this study. Option 2 was analyzed and modeled during a previous water system study and was found to be infeasible because it lowered the suction-side pressure of the booster pumps too much during a peak-hour demand. For this reason, Option 2 was not investigated further.

It is understood that the Tower Road Tank was constructed over 85 years ago on Frontage Road to provide the capability to expand the water system westward. As the Tower Road Tank operates today, it is hydraulically distant from the distribution system and provides little benefit during high demand periods. There is also a risk if the main supply line to the Tower Road Tank were to break under the Edens Expressway. Furthermore, the only water main between the Tower Road Tank and the distribution system is located under an environmentally sensitive area called the Skokie Lagoons. Because of these reasons, the Village should consider relocating the elevated water storage tank and thus would make Option 1 the most feasible option.

C. Peak Hour-Domestic and Fire Flow

The model was operated under peak hour demands when determining available fire flows throughout the system at each model junction. The estimated available fire flow, which was based on a minimum 20 psi residual pressure threshold, ranged from less than 500 gpm to greater than 4,000 gpm, as shown by the available fire flow contours generated by the model in Figure 1.02-2. Typically, the available fire flow will be highest near elevated storage, booster stations, and large-diameter transmission mains. The Village’s desired level of service goals are to provide a minimum of 500 gpm fire flow availability for residential areas and 2,000 gpm fire flow availability for commercial areas.

While fire flows below 500 gpm (red contoured areas) are below recommended minimums for residential areas, they regularly occur along dead-end mains. Lower fire flows in these areas suggest a



EXISTING PEAK HOUR PRESSURE CONTOURS

WATER DISTRIBUTION SYSTEM PLAN
VILLAGE OF GLENCOE
COOK COUNTY, ILLINOIS

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FIGURE 1.02-1
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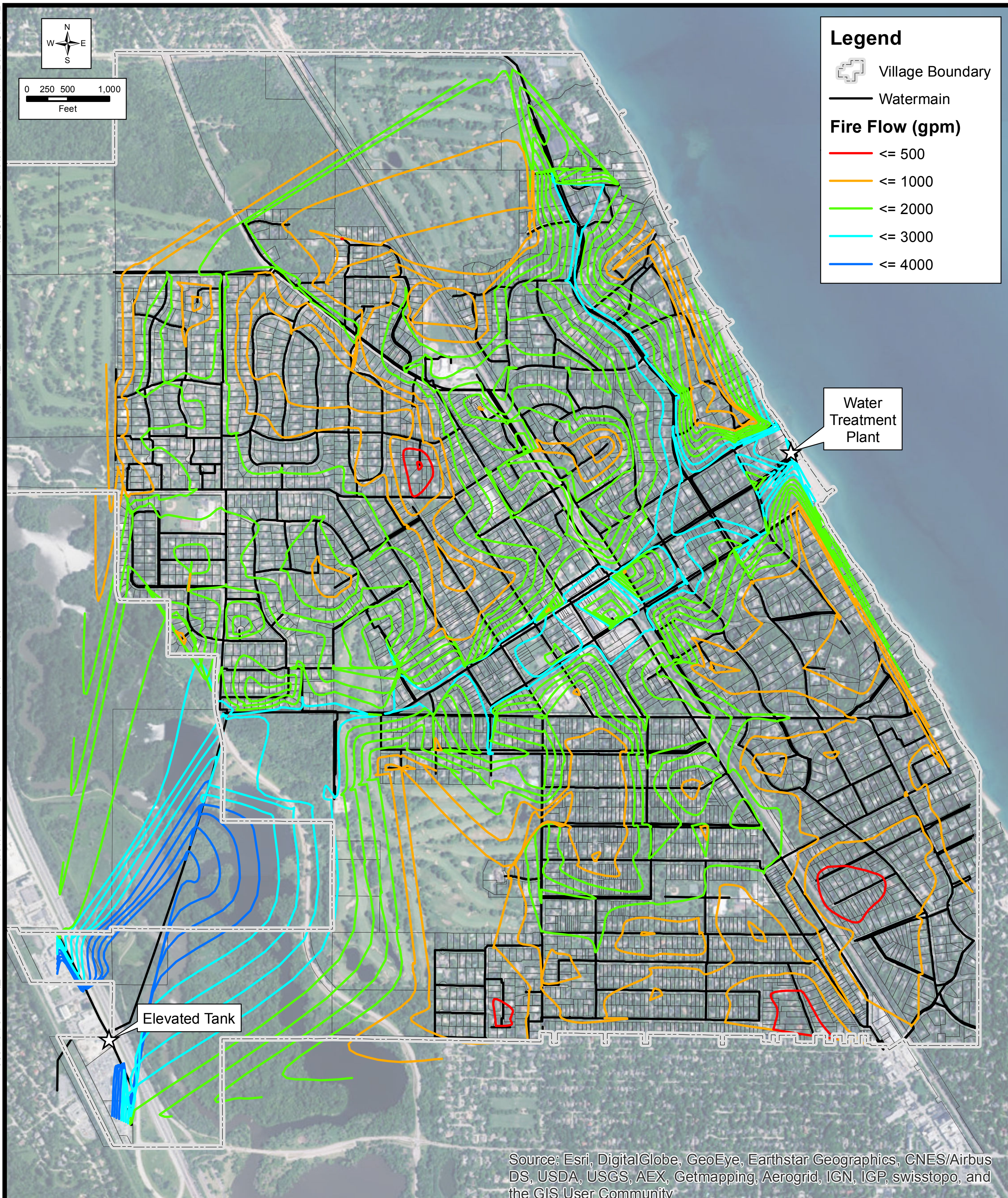

Path: S:\MAD\1400--1499\1410\017\Data\GIS\Figures\Figure 1.02-2 Fire Flow 8x11.mxd User: justinbi Date: 11/13/2015 Time: 9:01:35 AM

EXISTING PEAK HOUR AVAILABLE FIRE FLOW CONTOURS

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VILLAGE OF GLENCOE
COOK COUNTY, ILLINOIS

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FIGURE 1.02-2
1410.017

need to (1) provide additional water main looping in the distribution system, (2) install larger or new replacement distribution mains (minimum 8-inch diameter), or (3) have fire response teams connect to multiple hydrants during an emergency. Areas of deficient fire flows caused by either undersized mains or dead-end mains will addressed through water main improvements to increase available fire flow above recommended minimums. These improvements will be included in the 20-year capital improvement plan (see Section 3).

D. Main Break and Failure Data

The Village supplied main break mapping and break history data for review as part of this study. Data regarding water main material, age of pipe, and soil conditions was not reviewed. Figure 1.02-3 shows the general locations of water main breaks within the Village recorded in the Village's GIS system from 2000 to 2015. Spreadsheet data summarizing water main break date, location, and repair method was also supplied. Water main breaks can occur for several reasons including:

1. Age and condition of water main material.
2. Pressure surges or spikes within the distribution system.
3. Local soil conditions.
4. Climate conditions and changes.
5. Water main joint failure.

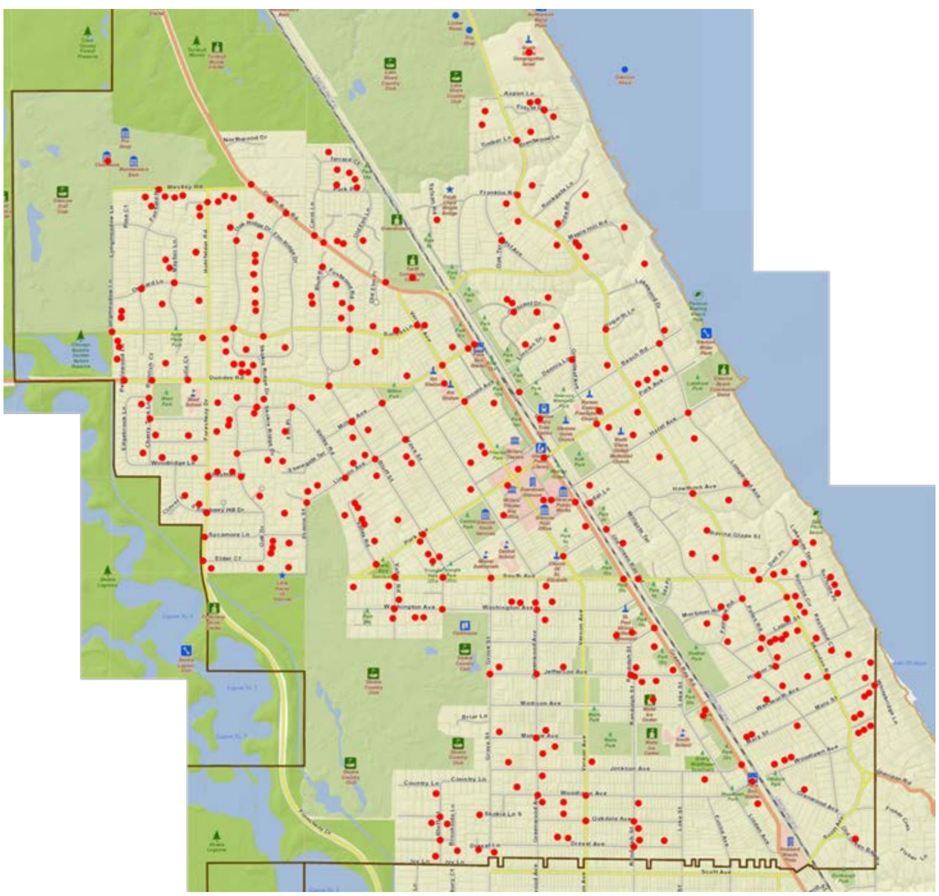


Figure 1.02-3 Village Main Break Data Summary (2000-2015)

Figure 1.02-4 displays the number of water main breaks from 2000 to 2015. Over this period, the average number of water main breaks within the distribution system was approximately 32 per year within the water system, which consists of 58 miles of installed water main. This average represents approximately 56 breaks per 100 miles of water main per year. In comparison, the American Water Works Association (AWWA) water distribution system integrity data collected from 153 utilities throughout the United States (AWWA Benchmarking Performance Indicators for Water and Wastewater Utilities, 2005) indicates that 56 breaks per 100 miles experienced by the Village is between the median value (43.6 breaks and leaks per 100 miles) and 75th percentile (78.7 breaks and leaks per 100 miles). While main break trends appear not to be increasing year over year, the higher average number of breaks per year indicates that the Village should consider prioritizing water main replacement based on water main failures along with the need to address other system deficiencies, as identified in Section 3.

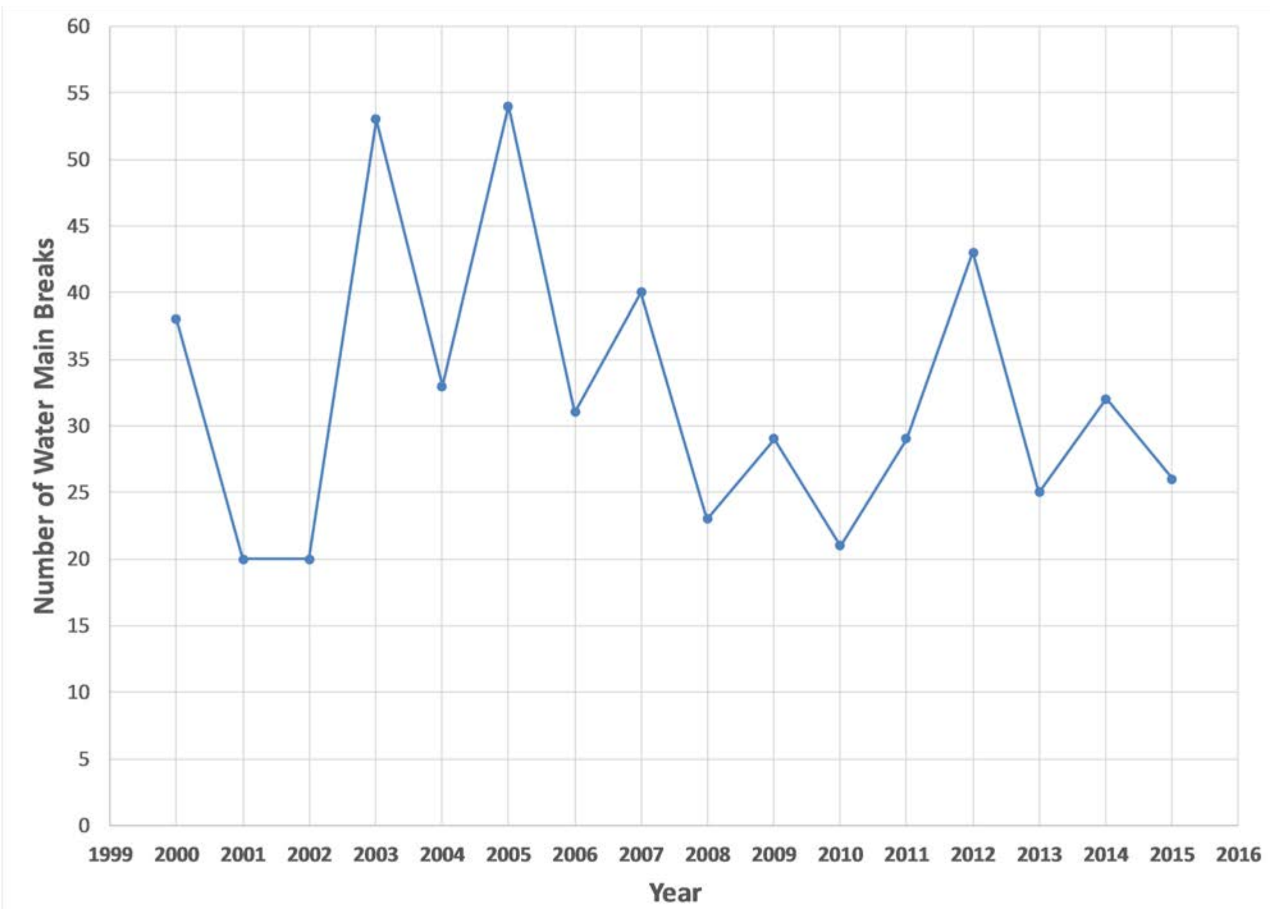


Figure 1.02-4 Annual Water Main Break Summary (2000 to 2015)

Although water main age was not reviewed as part of this study, it is important to understand how the age of water main plays a part in main breaks. Water main installation can generally be broken into three installation periods, the 1800s, 1900 to 1945, and post-1945. Within these periods, several types of processes were used to construct water main. The oldest pipe, dating back to the late 1800s, was created using a process called sand casting, also known as “thick wall” cast iron pipe. Pipes were cast vertically inside of a sand mold where molten cast iron was poured into the mold and slowly cooled until the pipe could be removed. These pipes generally have an average life of approximately 120 years. Because of technological innovations, such as spin-casting, cast iron pipe installed from the 1920s through the 1940s has an average life of approximately 100 years. Spun-cast pipe was produced by introducing molten cast iron into spinning horizontal molds, causing the metal to be evenly distributed about the surface of the mold. The same spin-casting pipe method is used today to create ductile iron piping. Continued technological innovations and material changes of cast iron pipes installed after World War II in the 1940s through 1950s produced an average life of approximately 75 years or less. Because of the changes in the casting process throughout the years, pipe manufactured and installed between the late 1800s and the 1950s may still be in service with similar average remaining life expectancy.

SECTION 2
WATER SYSTEM OPERATIONS AND RECOMMENDATIONS

2.01 ELEVATED TANK SUPPLY MAIN REINFORCEMENT

The Tower Road tank is connected to the water distribution system by a combination 10-inch and 14-inch water main starting at the area just south of Elder Court and west of Forestway Drive. The connecting supply main crosses under the Skokie Lagoons, an environmentally sensitive area adjacent to the Village limits, as well as the Edens Expressway. According to the Village's historical records, this water main predates the Skokie Lagoons and the Edens Expressway. Figure 2.01-1 shows the existing water main and other water system components.

The feasibility of strengthening the connection between the Tower Road Tank and water distribution system was evaluated for the following reasons:

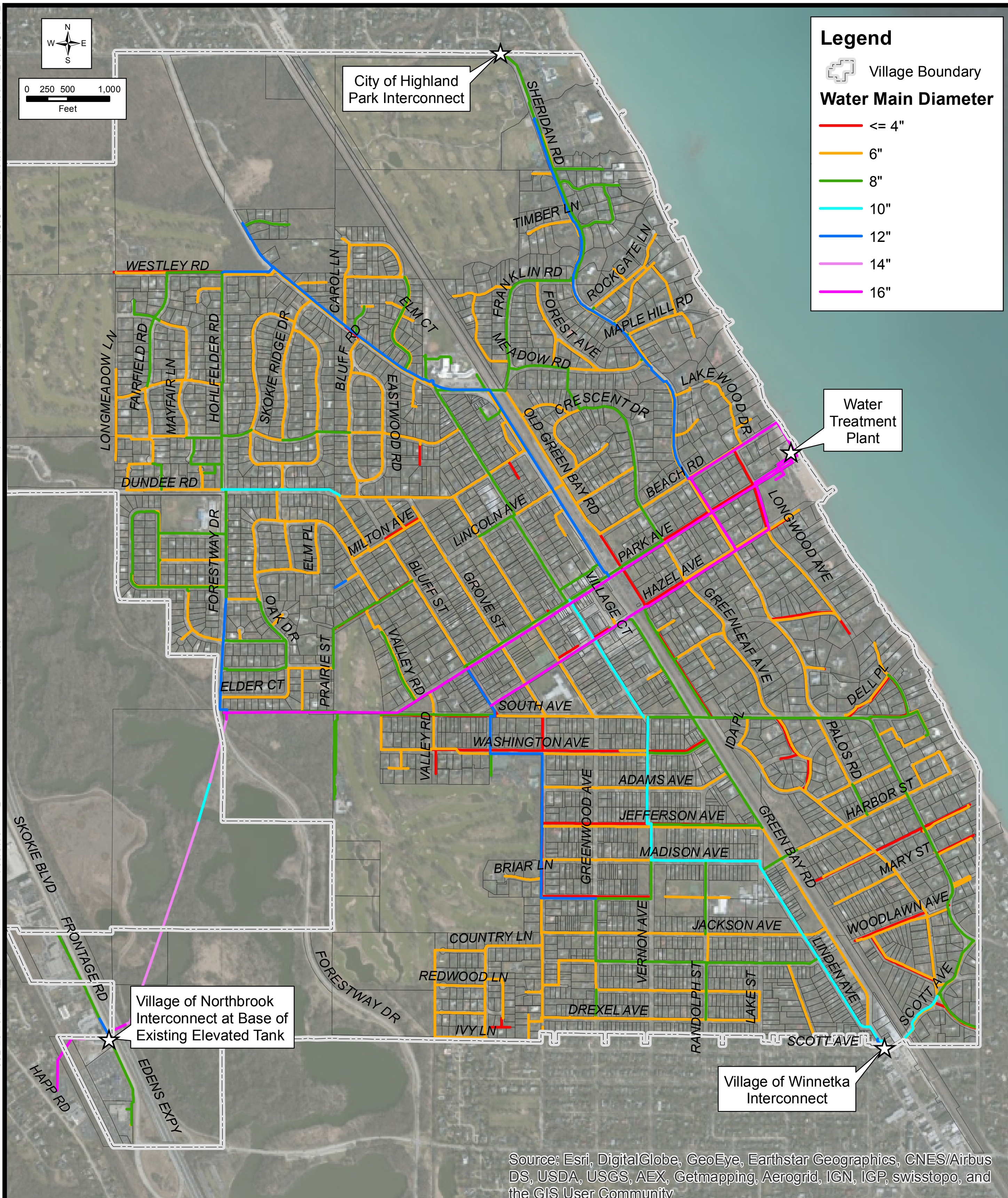
1. Provide redundancy for the existing critical water main.
2. Improve the operation during possible future emergency interconnection or emergency supply of water to and from Northbrook.
3. Improve water system operations during power outage conditions.

Two supply main reinforcement routes were selected for the analysis, as shown in Figure 2.01-2. The first route is along the existing 10-inch and 14-inch main under the Skokie Lagoons, connecting to the existing 16-inch main on each end. The second route includes extending transmission main south to Tower Road, east on Tower Road to Forestway Drive, then north along Forestway Drive to the existing 16-inch connection near Elder Court. Several scenarios were completed for these two potential routes and included varying transmission main sizes and eliminating the existing connection. The model was operated in a similar manner to the pressure and available fire flow analyses by using the peak-hour demand condition. Table 2.01-1 shows the results of each model run, starting with the results from the existing distribution system simulation.

Route Location	Connecting Main Size (in)	Eliminate Existing Main	High Lift Pump Output (gpm)	Elevated Tank Output (gpm)	Green Bay Transducer Pressure (psi)
None (Existing)	N/A	No	5,464	758	34.2
Skokie Lagoon	16	No	5,346	876	35.4
Skokie Lagoon	16	Yes	5,372	841	35.1
Skokie Lagoon	24	Yes	5,326	887	35.5
Tower Road	16	No	5,361	860	35.2
Tower Road	16	Yes	5,436	775	34.4
Tower Road	24	Yes	5,335	878	35.5

Table 2.01-1 Existing Tank Supply Main Model Results

While the redundant Skokie Lagoon transmission main improvements would be the preferred hydraulic option, the installation of this transmission main may prove difficult, if not impossible, to construct given the Forest Preserves control of the property. Using the Tower Road to Forestway Drive routing will provide a similar connectivity between the existing elevated tank and distribution system, with the need to obtain easements from the Illinois Department of Transportation (I-94 crossing) and Northfield and Winnetka along Tower Road.



WATER DISTRIBUTION SYSTEM

WATER DISTRIBUTION SYSTEM PLAN
VILLAGE OF GLENCOE
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FIGURE 2.01-1
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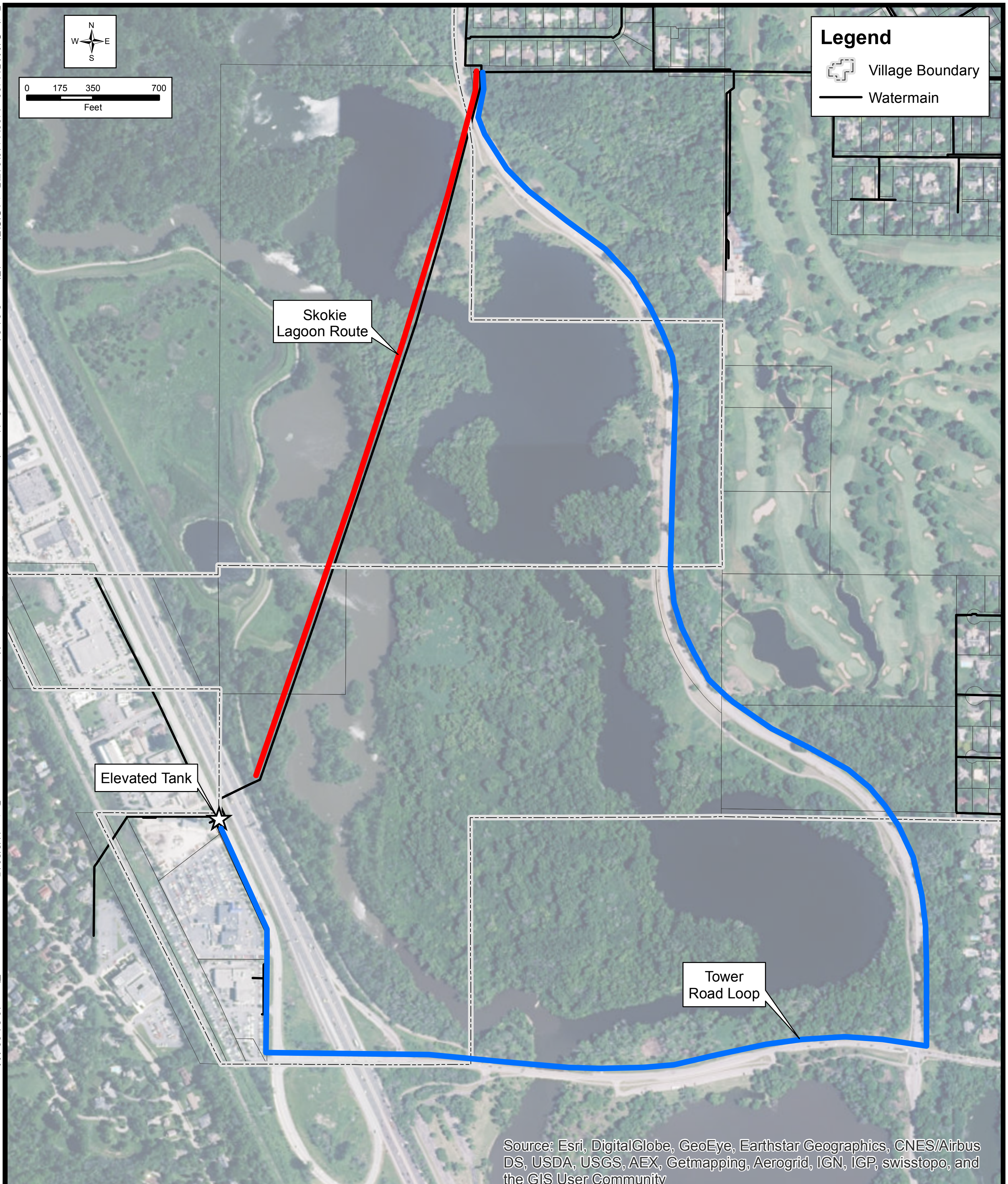

Path: S:\MAD\1400--1499\1410\017\Data\GIS\Figures\Figure 2.01-2 Improvements 8x11.mxd
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ELEVATED TANK CONNECTING MAIN IMPROVEMENTS

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FIGURE 2.01-2
1410.017

Based on the results of the modeling, a 16-inch water main along the Tower Road route is recommended to improve the Tower Road Tank and water distribution performance. Construction of the main would include approximately 11,000 feet of 16-inch transmission main with 1,000 feet of directional drilling under I-94 with casing pipe. The opinion of probable cost for this water main is approximately $6,325,000 and includes a 15 percent contingency for technical services and a 40 percent construction contingency. This would provide redundancy to the existing main and slightly improve system performance. However, compared to the cost of the improvement, the benefit supplied by this connecting main improvement is minimal because of the existing elevated tank location which limits hydraulic efficiency.

2.02 POTENTIAL ELEVATED TANK REPLACEMENT ANALYSIS

A. Location Analysis

To better improve pressure, available fire flow, system operation during power outage conditions, and the capability of elevated storage to provide water during peak demand periods, alternative elevated tank locations were modeled. Figure 2.02-1 shows the locations chosen, including at the existing elevated tank, near the intersection of Forestway Drive and Elder Court, and near the intersection of Green Bay Road and Northwood Drive.

The highest elevation within the system, as indicated by Village-provided digital elevation models, is 689 feet AMSL and is located in the northwest part of the Village near the intersection of Green Bay Road and Skokie Ridge Drive. To maintain 40 psi at this location, an overflow level of 781.4 feet AMSL is required from a static pressure perspective, ignoring any system loss between the elevated tank and the high point in the system. All scenarios were modeled with the proposed elevated tank overflow level set at 795 feet AMSL with the initial water level set at 785 feet AMSL to simulate the elevated tank operating within a normal operating range and accounting for friction loss between the elevated tank and the high point.

The model was operated in a similar manner to the pressure and available fire flow analysis using the peak-hour demand condition. Table 2.02-1 shows the results of each model run for the alternate elevated tank location analysis, starting with the results from the existing distribution system simulation. To increase the hydraulic strength of the distribution system, transmission main improvements were modeled for the Northwood Drive Area location. Improvements for the Northwood Drive site included replacing the 12-inch transmission main on Green Bay Road from Park Avenue to Northwood Drive with a 16-inch transmission main.

Path: S:\MAD\1400--1499\1410\017\Data\GIS\Figures\Figure 2.02-1 Replacement Locations 8x11.mxd

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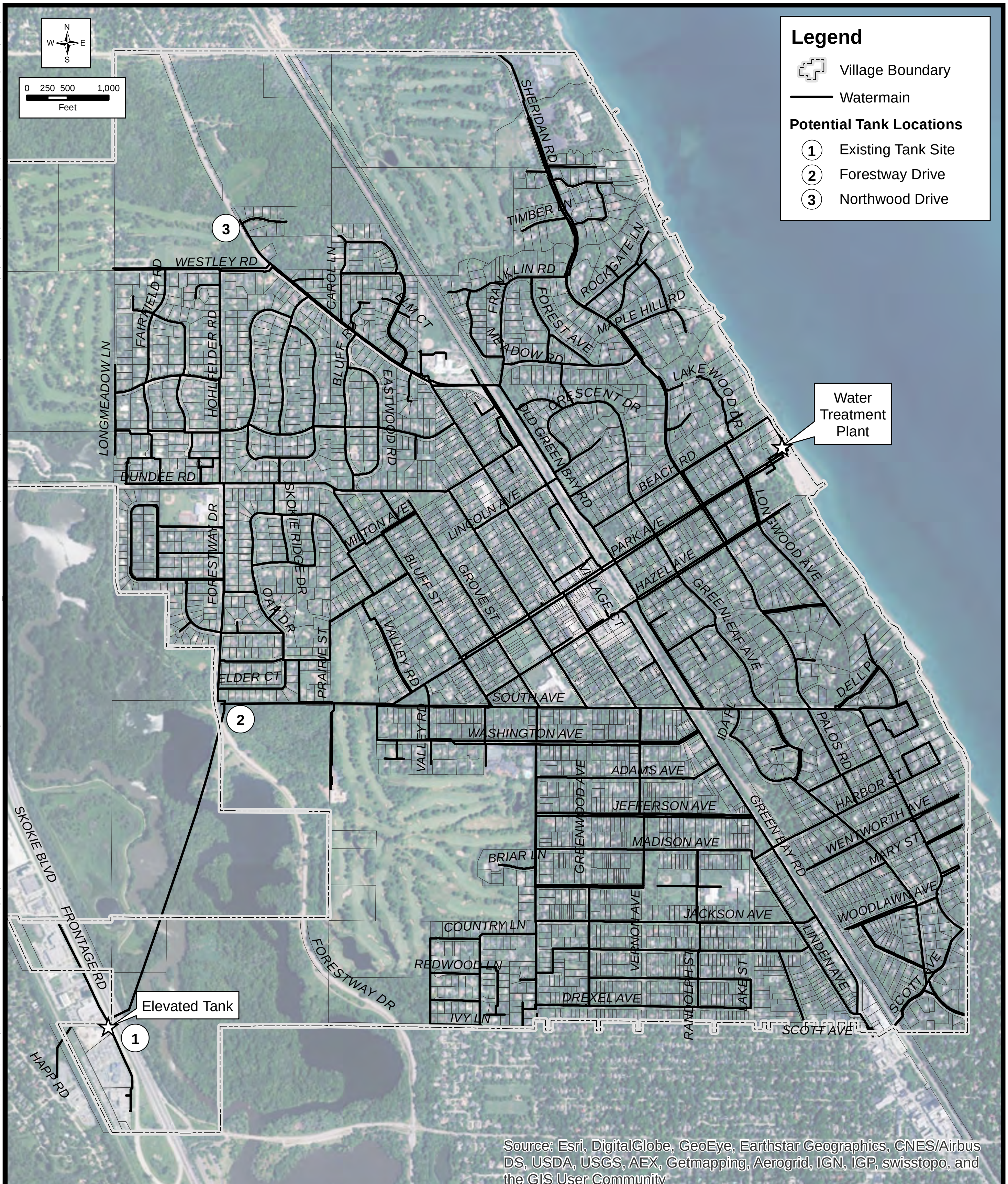

POTENTIAL ELEVATED TANK REPLACEMENT LOCATIONS

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FIGURE 2.02-1
1410.017

New Elevated Tank Location	High Lift Pump Output (gpm)	Elevated Tank Output (gpm)	Green Bay Transducer Pressure (psi)	Estimated Tank Height (feet)
Frontage Road (Existing Location)	5,464	758	34.2	155
Frontage Road (Existing Location - New Tank)	5,161	1,060	37.2	166
Forestway Drive/Elder Court Area	4,883	1,338	40.0	166
Northwood Drive Area	5,026	1,196	41.3	112
Northwood Drive Area with Improvements	4,886	1,336	42.4	112

Table 2.02-1 Elevated Tank Replacement Model Results

Based on the results of the modeling, it is recommended to consider constructing a new elevated tank near the Forestway Drive/Elder Court area. This location provides the greatest benefit to the entire system with the least amount of improvements required. In addition, this proposed location eliminates the location and ongoing risk of the connecting supply main through the Skokie Lagoon area. The new proposed elevated tank was estimated to provide 1,338 gpm during a peak-hour demand event, which represents a 77 percent increase in available flow compared to the available from the Tower Road tank. With the proposed tank location, pressures within the northwest area of the Village were also able to maintain approximately 40 psi pressure.

B. Storage Capacity Analysis

The existing 2.0 MG reservoir on the east end of Park Avenue is a critical water system component because it provides storage for the WTP to meet the range of system demands in addition to meeting fire flow demands. As part of the 2015 *Water Supply Planning Report*, modifications to the existing reservoir were contemplated. The reservoir will need to be taken out of service for several months during construction of improvements to include bifurcation of the reservoir and a new booster pump station.

In light of these improvements, we evaluated the recommended elevated storage volume needed if the 2 MG reservoir was not in service during a maximum day demand of 5.4 mgd or 3,760 gpm. Because timing of the reservoir improvements is unknown at this point and because of the treatment capacity differences between the current and future WTPs, two separate storage capacity analyses were completed.

1. Existing WTP Treatment Capacity

This storage capacity section assumes the reservoir is out of service while the current WTP is in operation. The current treatment capacity of the WTP is 7.3 mgd or 5,069 gpm. The firm high lift pumping capacity, with High Lift Pump No. 4 out of service, is approximately 7.93 mgd or 5,500 gpm. If the reservoir is assumed to be out of service, any flow greater than the treatment capacity of the WTP would have to be removed from the clearwell. This is not recommended because of contact time disinfection requirements. Therefore, the capacity of the high lift pumps is assumed to be limited to that of the treatment capacity.

Figure 2.02-2 presents a graph of the projected hourly demands for the maximum day. Hourly peaking factors were obtained from tank level and high lift pump flow data from the Village's SCADA system.

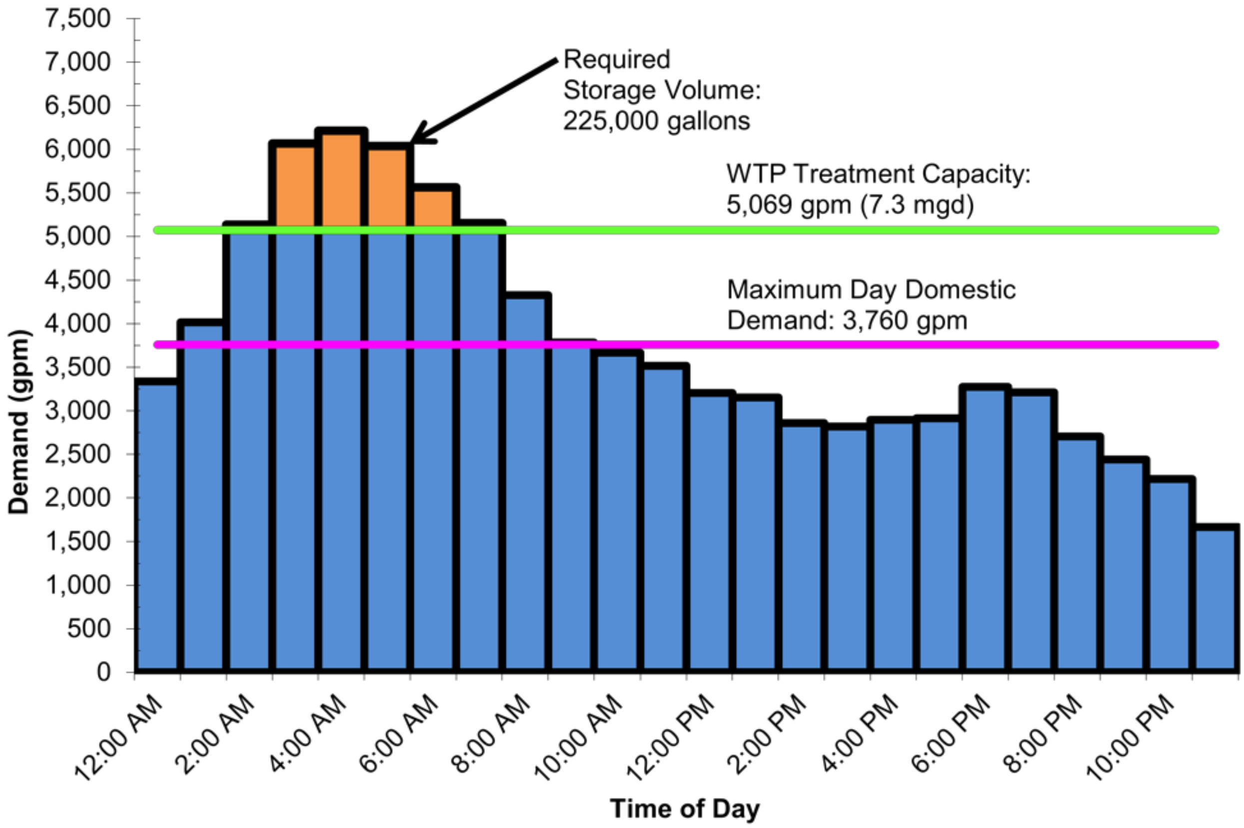

Figure 2.02-2 Projected Maximum Day Hourly Demands (7.3 mgd WTP Capacity)

Because there are periods of time where the projected hourly demand is greater than the WTP treatment capacity, storage must be used to satisfy water demands. Approximately 225,000 gallons of water are required from storage to meet projected hourly demands. Typically, 10 percent of the elevated storage, or 50,000 gallons, is needed for control and operational needs. Therefore, the total required elevated storage to meet maximum day domestic demands is 275,000 gallons. It is assumed this storage is removed from the 0.5 MG Tower Road Tank, leaving 225,000 gallons.

As described in the 2015 *Water Supply Planning Report*, the Village selected a fire flow of 1,500 gpm for 2 hours. When calculating available storage, a demand rate of 5,260 gpm (3,760 gpm domestic demand plus 1,500 gpm fire demand) for 2 hours must be satisfied to provide the targeted minimum fire protection. Because a fire can start at any time during the day, domestic use must be taken into account when calculating available capacity.

Maximum Day Demand	- 3,760 gpm
Fire Demand	- 1,500 gpm
WTP Production Rate (7.3 mgd)	+ 5,069 gpm
0.5 MG Elevated Tank Contribution*	+ 1,875 gpm
Total (Reserve)	**+ 1,684 gpm**

*Storage capacity = 225,000 gallons/120 minutes

During a 120-minute fire event, the system is projected to have a storage capacity reserve of approximately 202,000 gallons (1,684 gpm x 120 minutes). This indicates that the current elevated storage capacity of 0.5 MG is sufficient.

2. Existing WTP Treatment Capacity

This section assumes the reservoir is out of service while the future WTP is in operation. The future treatment capacity of the WTP is 6.0 mgd or 4,167 gpm. As described previously, the capacity of the high lift pumps is assumed to be limited to that of the treatment capacity. Figure 2.02-3 presents a graph of the projected hourly demands for the maximum day.

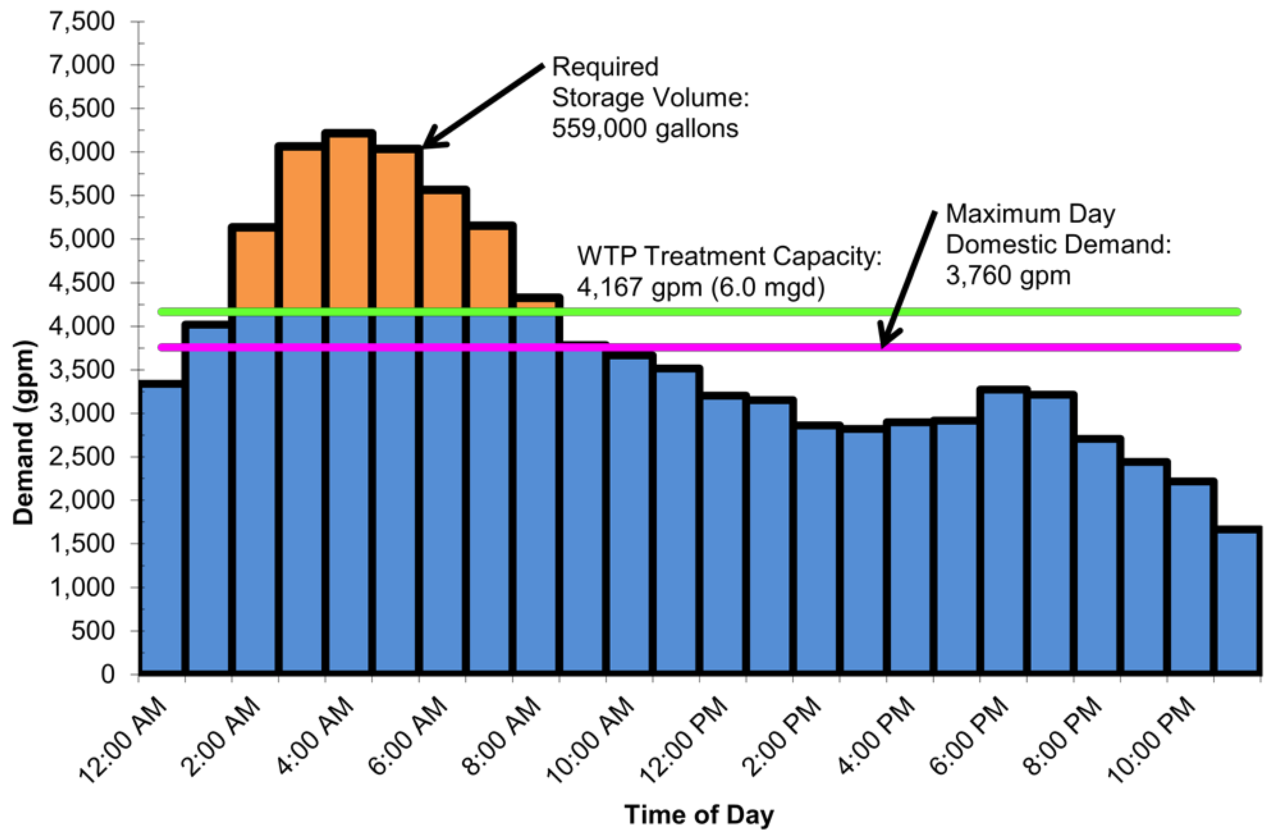

Figure 2.02-3 Projected Maximum Day Hourly Demands (6.0 mgd WTP Capacity)

Because there are periods of time where the projected hourly demand is greater than the WTP treatment capacity, storage must be used to satisfy water demands. Approximately 559,000 gallons of water are required from storage to meet projected hourly demands. Typically, 10 percent of the elevated storage, or 50,000 gallons, is needed for control and operational needs. Therefore, the total required elevated storage to meet maximum day domestic demands is 609,000 gallons. It is assumed this storage is removed from the 0.5 MG Tower Road Tank, leaving a theoretical -109,000 gallons of storage.

When calculating available storage, a demand rate of 5,260 gpm (3,760 gpm domestic demand plus 1,500 gpm fire demand) for 2 hours must be satisfied to provide the targeted minimum fire protection. Because a fire can start at any time during the day, domestic use must be taken into account when calculating available capacity.

Maximum Day Demand	- 3,760 gpm
Fire Demand	- 1,500 gpm
WTP Production Rate (6.0 mgd)	+ 4,167 gpm
0.5 MG Elevated Tank Contribution*	- 908 gpm
Total (Deficit)	**- 2,001 gpm**

*Storage capacity = -109,000 gallons/120 minutes

During a 120-minute fire event, the system is projected to have a storage capacity deficit of approximately 240,000 gallons (2,001 gpm x 120 minutes). This indicates that the current elevated storage capacity of 0.5 MG is not sufficient.

3. Capacity Analysis Summary

Based on the timing of the reservoir improvements, the size of the new elevated tank changes. If the reservoir were to be taken down with the current WTP (7.3 mgd) in service, it is recommended that the proposed elevated tank match size of the 0.5 MG Tower Road Tank. The opinion of probable cost for a 0.5 MG elevated tank is approximately $2,600,000 and includes a 15 percent contingency for technical services and a 20 percent construction contingency. If the reservoir were to be taken down with the future WTP (6.0 mgd) in service, it is recommended that the proposed elevated tank size be at least 0.75 MG. The opinion of probable cost for the a 0.75 MG elevated tank is approximately $3,645,000 and includes a 15 percent contingency for technical services and a 20 percent construction contingency.

2.03 OPERATIONAL EFFICIENCY IMPROVEMENTS

The Village maintains water system pressure through the use of four high lift pumps located in the basement of the WTP. These four pumps represent the largest, if not the only, electrical demand within the distribution system operated by the Village. These pumps are the main focus for energy savings and optimization, as previous studies have identified other operational costs savings within the water treatment plant. Table 2.03-1 summarizes the design characteristics of the high lift pumps.

Pump No.	Design Flow (gpm)	Design Head (feet)	Horsepower	Design Pump Efficiency	Motor Control
1	700	205	100	<71%	Automated, VFD
2	1,750	205	150	87%	Automated, VFD
3	2,436	207	200	90%	Manual, Soft-start
4	3,250	207	250	85%	Manual, Soft-start

Table 2.03-1 High Lift Pump Design Conditions

The Village's *Water Plant SCADA System Operating Manual* (March 2001) describes the basic theory of operation for the high lift pumps. Manual operation of all four pumps is based on operator's experience and observation of the distribution system.

During low to average demand conditions, operators will typically operate either Pump Nos. 1 or 2 using the automated variable frequency drive (VFD) control to maintain a consistent system pressure. As demand increases, additional pumps are brought online.

To balance water production and system demands with high lift pump station operation during above average day conditions, the discharge valve on Pump Nos. 3 or 4 may be throttled while either Pump Nos. 1 or 2 are in operation. As an example of operation, there are operational conditions that may require operators to throttle back Pump No. 3 at a rate of one-quarter of its flow capacity while Pump No. 1 is in operation.

During summer and maximum day flows, operators will most likely operate Pump Nos. 3 and 4 together.

For the purposes of this study, wire-to-wire efficiency was not evaluated. These tests are beneficial in determining if the existing impellers have worn and were becoming more inefficient compared to their design conditions. Regular maintenance and pump testing, on the order of every 1 to 3 years, are standard preventive measures toward maintaining pump performance.

In order to understand the efficiency of operations, the available pump curve data was used to generate probable cost per one-thousand gallons ($/kgal) of water production with each pump at constant speed (100 percent speed) at $0.08 per kilowatt hour (kWh) (Glencoe Electrical Utility Bill, 2015). Figure 2.03-1 demonstrates the cost efficiency of Pump Nos. 1, 2, 3, and 4 over the range of possible flow conditions. Depending on demand, this figure indicates that operators should generally attempt to operate Pump Nos. 1, 2, 3, and 4, consecutively, to supply flow to the Village to provide the most efficient pumping conditions.

As an example of how to use Figure 2.03-1, one could consider the difference between operating conditions where throttling back of Pump No. 3 occurs with Pump No. 1 at 100 percent speed or selecting a different pump. If the operator was required to set a flow of 2,000 gpm, where Pump No. 1 is at 1,500 gpm and Pump No. 3 is throttled to 500 gpm, the probable operating cost of the pumps is approximately $0.056 per kgal and $0.140 per kgal, respectively, for a total of $0.196 per kgal. The figure would indicate that Pump No. 2 may operate at the most efficient production rate of $0.055 per kgal for 2,000 gpm.

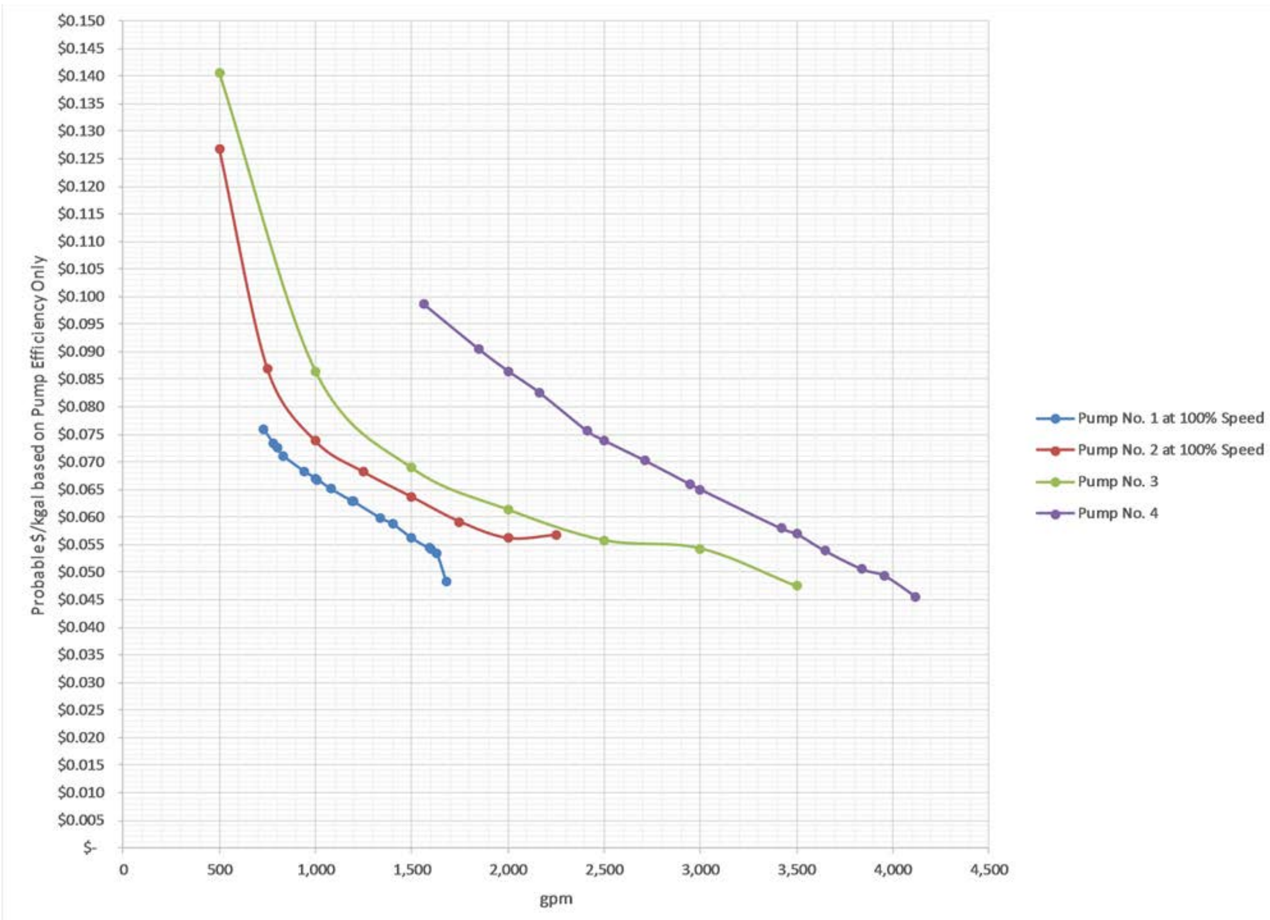

Figure 2.03-1 High Lift Pump Probable Cost of Operation Compared to Flow

However, operators must continue to rely on operational experience and what pumping options they have available:

1. In selecting Pump No. 2, changes in system pressure and demand at flows beyond 2,000 gpm may cause the pump to cavitate, requiring additional maintenance costs to replace the pump impeller over time.

2. Another option would be to run Pump No. 3 alone with slight throttling. If system head allowed Pump No. 3 operation near 3,000 gpm, throttling the pump back to 2,000 for an additional increase of $0.007/kgal, or $0.062/kgal, is still less expensive compared to operating Pump Nos. 1 and 3 under throttled conditions. This approach does not include the ability to use the VFD on Pump No. 1 to reduce flow should system demands decrease.

3. Operating Pump No. 4, while being the largest pump in service, throttled back would be able to operate within its pump curve at 2,000 gpm. While still not as efficient as

Pump No. 3 partially throttled, it could potentially be operated 2,000 gpm as a more efficient option ($0.087 per kgal).

4. Given the stigma of throttling a pump to operate, installing a VFD drive on Pump No. 3 may be an option the Village may want to implement. The probable cost to install a new VFD with new wiring, motor, and control programming (Lead-Lag-Lag arrangement) would represent a probable cost of approximately $127,000, including general conditions (GCs), technical services (15 percent), and contingency (20 percent).

When Pump Nos. 1 and 2 are in VFD operation, Figure 2.03-2 demonstrates the range of pump operational cost using the VFD drives on Pump Nos. 1 and 2. The dashed line represents the highest pump efficiency as the pump speed is adjusted through the range of VFD speeds. Pump No. 1 is capable of running normally between 94 to 100 percent speed. Pump No. 2 is capable of running normally between 92 to 100 percent speed. Both pumps are very similar throughout the range of VFD speeds from a probable cost of production ($/kgal) standpoint and are sized to handle minimum to average day conditions based on historical demands.

The current supervisory control and data acquisition (SCADA) system operations only allow one of the VFD driven pumps to operate under setpoint control as a lead pump. SCADA control modifications could be made to control the pumps to operate one or both pumps more efficiently. In this scenario, the controls would be modified to allow a Lead-Lag arrangement. The lead pump would operate on the 40 psi setpoint until it reached a maximum speed setpoint (92 percent to 100 percent) for a period of time, after which the second lag pump could be brought online with the lead pump. Both pumps operating together would be operated at the same speed setpoint to maintain the system pressure setpoint. If the minimum speed setpoint (92 percent to 94 percent) were reached for a period of time, the lag pump would then be taken out of service, allowing the lead pump to continue to operate. This basic control scenario allows Pump Nos. 1 and 2 to fully use the VFD capabilities and possibly improve operations, i.e., fewer operator decisions in making pump changes. To reduce water hammer in this arrangement, programming to allow the pump to run underspeed for 2 to 3 minutes after start-up and then ramp up to normal operating speed (92 percent to 100 percent) should also be implemented with this upgrade to mimic current operations. The opinion of probable cost for implementing the VFD control upgrades is approximately $25,000.

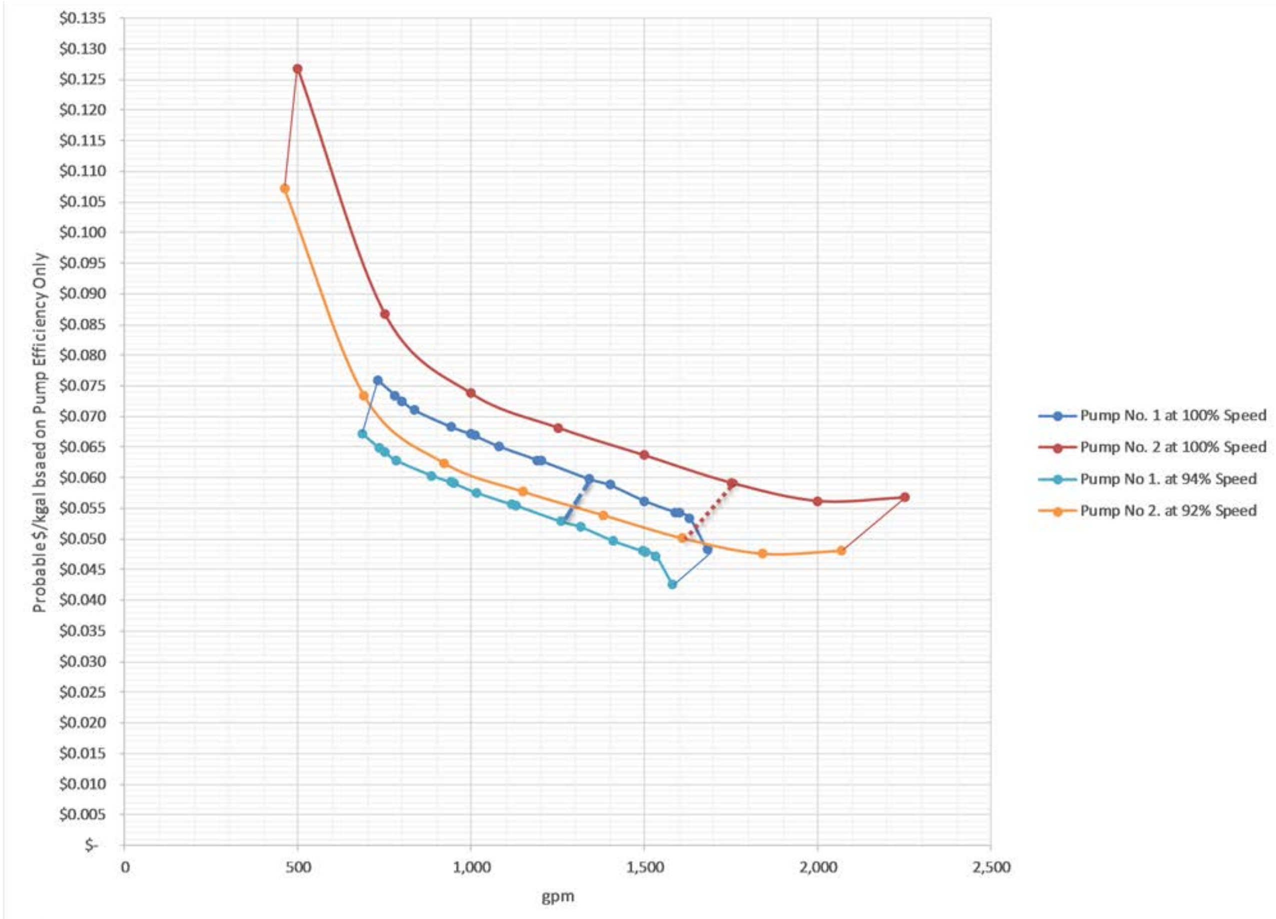

Figure 2.03-2 VFD Operated Pump Nos 1 and 2 Probable Cost of Operation Compared to Flow

It is difficult to forecast what actual cost savings could be achieved using the information above because operators have a variety of pump options and maintenance needs to meet water system demands in a given year. In order to achieve operational savings, the following options are given for consideration:

1. Minimize the number of pump starts during a given day–This step reduces the amount of in-rush power (kWh) required to start a pump going to the motor of a given pump.

2. Select pumps in order of efficiency based on system demands–This step helps the Village use the most efficient pump. As demands increase and decrease, the Village typically steps from Pump Nos. 1 and 2 up to Pump Nos. 3 and 4 and back again, especially from May to October of a given year when demand is higher.

3. Conduct annual pump tests to confirm pump operation–This step verifies the operation of the pump against published pump curve data to check for pump wear or

deterioration. A standard pump test can take up to an hour to perform on each pump and may signal that pump maintenance or impeller replacement is necessary.

4. Monitor pump performance for changes in temperature and amperage–This step uses thermal imaging equipment and VFD data to check for issues with the motor or VFD or both. Increases in motor temperature or amperage above normal may be an indicator of impending motor or VFD drive failure.

5. Continue to track electrical costs with pump operations and system demands–This step allows for comparison of previous years' data to evaluate fluctuations in annual operations. Tracking energy consumption and pump usage is useful to help understand if performance is really improving.

6. Continue water main flushing and replacement within the distribution system–This step maintains the pipe materials and condition by helping to minimize friction loss along the length of the water main, especially along the largest system mains between the water treatment plant and elevated storage. While a minor energy savings consideration for a system of this size, maintaining the hydraulic conditions of the pipes and minimizing friction losses within the system is good standard practice.

Consistently making good operational decisions, within the range of normal operating conditions, can lead to potential operational cost savings. Given the current efficient operations of the WTP, potential operational cost savings would be minimal and would not pay for the proposed capital improvements at this time. The operational efficiency of future improvements will need to be considered as new WTP and distribution system components are considered.

SECTION 3
CAPITAL IMPROVEMENTS PLAN

3.01 CAPITAL IMPROVEMENT PLAN SUMMARY

A Capital Improvement Plan (CIP) was created for years 2018 through 2038 using first quarter 2016 dollars and was based on the analysis of the existing distribution system under peak demand, water main break data, and input from Village staff. It is recommended to coordinate the projects with other Village infrastructure improvements, such as sanitary sewer, storm sewer, and streets. Particular detail should be given to the order and timing of street rehabilitation and resurfacing projects. The prioritized CIP is shown in Table 3.01-1. Projects were assigned a year by priority using ratio of breaks per 100 feet of pipe with the highest ratio being completed first. Areas with deficient fire flows were also given a high priority. Costs for major capital improvement projects, such as the Tower Road Tank supply main reinforcement and elevated tank replacement, were not included in the prioritized list because of the uncertainty of the project timing.

The budgets for the projects were determined by reviewing bid results from neighboring communities within the past three years of utility construction projects. The total opinion of probable cost includes construction cost and a 25 percent contingency. The contingency value is meant to cover miscellaneous costs such as erosion control, traffic control, pipe fittings, connections to existing pipes, and unsuitable soils. The total cost does not include engineering design or construction administration services.

Figure 3.01-1 shows the proposed distribution system improvements.

Map ID	Location	Year Starting
A	Sunset Lane (Sunset to south end)	2018
B	Wentworth Avenue/Mary Street Loop	2018
C	Mary Street (Sheridan to east end)	2018
D	Oak Drive (Sycamore to Elder)	2018
E	Brookside Lane/Bluff Street Loop and Service Relocation to Ivy Lane	2019
F	Euclid Avenue (Woodlawn to south end of main)	2019
G	Lincoln Drive (Crescent to northeast end of road)	2019
H	Longmeadow Lane Loop (Westley to Westwood)	2020
I	Forestway Drive (Strawberry Hill to Chestnut)	2020
J	Longwood Avenue (Hawthorn to southeast end of road)	2020
K	Pebblewood Lane (Dundee to north end of road)	2021
L	Whitebridge Hill Road (Sheridan to east end)	2021
M	Hawthorn Avenue (Sheridan to Longwood)	2021
N	Dundee Road (Green Bay to 10" on Dundee)	2022
O	Sunrise Circle (Sheridan to southeast end of road)	2022
P	Skokie Ridge Drive (Oak/Elm Ridge to Sunset)	2023
Q	Sheridan Road (Hazel to South)	2024
R	Dell Place (Sheridan to Lakeside)	2024
S	Sheridan Road/Scott Avenue (South to Old Green Bay)	2025
T	Greenwood Avenue (Oakdale to Monroe)	2026
U	Lapier Street (Palos to Sheridan)	2026
U-1	Hawthorn Avenue (Green Bay to Greenwood)	2026
V	Westley Road (Hohlfelder to west end of road)	2027
W	Keystone Court (Harbor to north end of road)	2027
X	Milton Avenue (Bluff to Grove)	2028
Y	Bluff Street (Milton to Lincoln)	2028
Z	Lincoln Avenue (Prairie to Valley)	2028
AA	Valley Road (South to south end of road)	2029
BB	Bluff Road Main Abandonment (Park to South)	2029
CC	Sycamore Lane (Oak to Prairie)	2029
DD	Oak Drive (Valley to Stonegate)	2029
EE	Crescent Drive (Lincoln to east end of road)	2030
FF	Park Avenue (Sheridan to Longwood)	2030
GG	Longmeadow Lane Loop (Orchard to Pebblewood)	2031
HH	Eastwood Road (Bluff to Sunset)	2031
II	Hohlfelder Road (Westley to 230' south of Orchard)	2032
JJ	Bluff Road (Country to Ivy)	2033
KK	Apple Tree Lane (Cherry Tree to Forestway)	2033
LL	Orchard Lane/Longmeadow Lane Loop (Hohlfelder to Westwood)	2034
MM	Mayfair Lane (Orchard to Westwood)	2035
NN	Woodlawn Avenue Abandonment (Old Green Bay to west end of road)	2035
OO	Mortimer Road Loop and Abandonment (Old Green Bay to Greenleaf)	2035
PP	Park Place (Carol to Old Elm)	2035
QQ	Drexel Avenue (Greenwood to Lake)	2036
RR	Oak Ridge Drive (Sunset to 1029 Oak Ridge)	2037
SS	Terrace Court (Old Elm to end of Carol)	2037
TT	Washington Avenue (Bluff to Vernon)	2038
UU	Monroe Avenue Abandonment (Grove to Vernon)	2038
VV	Washington Place Loop	2038

DISTRIBUTION SYSTEM CAPITAL IMPROVEMENTS PLAN

WATER DISTRIBUTION SYSTEM PLAN
VILLAGE OF GLENCOE
COOK COUNTY, ILLINOIS

STRAND ASSOCIATES®

FIGURE 3.01-1
1410.017

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Table 3.01-1 Water System Capital Improvements Plan

Map ID	Project	Main Diameter (inches)	Year Starting	Opinion of Probable Cost	Length (feet)	Comments
A	Sunset Lane (Sunset to south end)	6	2018	$83,800	290	Increase fire flow.
B	Wentworth Avenue/Mary Street Loop	8	2018	$175,800	650	Eliminate dead ends on Wentworth and Mary. Abandon 4-inch parallel main on Wentworth (4 service switchovers).
C	Mary Street (Sheridan to east end)	6	2018	$36,800	70	Abandon 4-inch parallel main (5 service switchovers). Extend 6-inch main to end of road and install hydrant.
D	Oak Drive (Sycamore to Elder)	8	2018	$92,000	300	Age and condition of pipe.
E	Brookside Lane/Bluff Street Loop and Service Relocation to Ivy Lane	8	2019	$82,300	230	Abandon 4-inch main (6 service switchovers) on Brookside. Increase fire flow. Eliminate dead end on Brookside.
F	Euclid Avenue (Woodlawn to south end of main)	8	2019	$291,700	930	Increase fire flow.
G	Lincoln Drive (Crescent to northeast end of road)	8	2019	$149,200	467	Increase fire flow.
H	Longmeadow Lane Loop (Westley to Westwood)	8	2020	$210,200	750	Eliminate dead end on Westley.
I	Forestway Drive (Strawberry Hill to Chestnut)	12	2020	$166,700	497	History of main breaks.
J	Longwood Avenue (Hawthorn to southeast end of road)	6	2020	$81,600	296	Increase fire flow.
K	Pebblewood Lane (Dundee to north end of road)	8	2021	$82,000	250	Age of condition of pipe. Looping system improvement.
L	Whitebridge Hill Road (Sheridan to east end)	8	2021	$149,800	510	History of main breaks.
M	Hawthorn Avenue (Sheridan to Longwood)	8	2021	$233,700	770	History of main breaks.
N	Dundee Road (Green Bay to 10" on Dundee)	12	2022	$801,900	2,450	Connect large-diameter main to each other. History of main breaks.
O	Sunrise Circle (Sheridan to southeast end of road)	8	2022	$113,000	390	History of main breaks.
P	Skokie Ridge Drive (Oak/Elm Ridge to Sunset)	8	2023	$451,700	1,457	History of main breaks. Poor soil and pipe conditions.
Q	Sheridan Road (Hazel to South)	16	2024	$1,232,500	2,900	History of main breaks. Extend transmission main to south end of Village. Increase fire flow to Hubbard Woods Plaza.
R	Dell Place (Sheridan to Lakeside)	8	2024	$220,500	760	History of main breaks.
S	Sheridan Road/Scott Avenue (South to Old Green Bay)	16	2025	$1,657,500	3,900	History of main breaks. Extend transmission main to south end of Village. Increase fire flow to Hubbard Woods Plaza.
T	Greenwood Avenue (Oakdale to Monroe)	8	2026	$319,700	1,100	History of main breaks.
U	Lapier Street (Palos to Sheridan)	8	2026	$159,700	539	Age and condition of pipe.
U-1	Hawthorn Avenue (Green Bay to Greenwood)	8	2026	$299,600	1,020	No main currently with long, lead service lines.
V	Westley Road (Hohlfelder to west end of road)	8	2027	$401,600	1,320	History of main breaks. Abandon 4-inch parallel main (6 service switchovers).
W	Keystone Court (Harbor to north end of road)	8	2027	$91,100	290	History of main breaks.
X	Milton Avenue (Bluff to Grove)	8	2028	$160,300	525	History of main breaks. Abandon 4-inch parallel main (no service switchovers).
Y	Bluff Street (Milton to Lincoln)	8	2028	$173,200	554	History of main breaks.
Z	Lincoln Avenue (Prairie to Valley)	8	2028	$193,100	588	History of main breaks. Abandon 4-inch parallel main (8 service switchovers).
AA	Valley Road (South to south end of road)	8	2029	$200,400	721	History of main breaks.
BB	Bluff Road Main Abandonment (Park to South)	--	2029	$28,000	678	Abandon 4-inch parallel main only (6 service switchovers).
CC	Sycamore Lane (Oak to Prairie)	8	2029	$163,700	547	History of main breaks.
DD	Oak Drive (Valley to Stonegate)	8	2029	$219,000	707	Age and condition of pipe.
EE	Crescent Drive (Lincoln to east end of road)	8	2030	$440,400	1,450	History of main breaks.
FF	Park Avenue (Sheridan to Longwood)	8	2030	$187,400	626	Parallel 16-inch to remain.
GG	Longmeadow Lane Loop (Orchard to Pebblewood)	8	2031	$277,700	961	Eliminate dead end on Longmeadow.
HH	Eastwood Road (Bluff to Sunset)	8	2031	$303,800	981	History of main breaks.
II	Hohlfelder Road (Westley to 230' south of Orchard)	8	2032	$497,800	1,570	History of main breaks.
JJ	Bluff Road (Country to Ivy)	8	2033	$296,600	1,050	History of main breaks.
KK	Apple Tree Lane (Cherry Tree to Forestway)	8	2033	$249,100	796	Age and condition of pipe.

Map ID	Project	Main Diameter (inches)	Year Starting	Opinion of Probable Cost	Length (feet)	Comments
LL	Orchard Lane/Longmeadow Lane Loop (Hohlfelder to Westwood)	8	2034	$680,600	2,350	Eliminate dead end on Longmeadow.
MM	Mayfair Lane (Orchard to Westwood)	8	2035	$191,100	626	Age and condition of pipe.
NN	Woodlawn Avenue Abandonment (Old Green Bay to west end of road)	--	2035	$37,500	899	Abandon 4-inch parallel main only (7 service switchovers). Reconnect 4-inch main on Glenwood into 6-inch main.
OO	Mortimer Road Loop and Abandonment (Old Green Bay to Greenleaf)	8	2035	$163,500	470	Eliminate dead end on Mortimer. Abandon 4-inch parallel main on Greenleaf and Fairview (8 service switchovers).
PP	Park Place (Carol to Old Elm)	8	2035	$218,000	710	History of main breaks. Poor pipe and soil conditions.
QQ	Drexel Avenue (Greenwood to Lake)	8	2036	$650,000	1,980	History of main breaks.
RR	Oak Ridge Drive (Sunset to 1029 Oak Ridge)	8	2037	$219,900	758	History of main breaks. Poor pipe and soil conditions.
SS	Terrace Court (Old Elm to end of Carol)	8	2037	$303,500	1,014	History of main breaks. Increase fire flow.
TT	Washington Avenue (Bluff to Vernon)	8	2038	$427,700	1,280	Abandon 4-inch and 6-inch parallel mains (25 service switchovers).
UU	Monroe Avenue Abandonment (Grove to Vernon)	--	2038	$45,200	1,329	Abandon 4-inch parallel main only (13 service switchovers).
VV	Washington Place Loop	6	2038	$78,900	350	Eliminate dead end on Washington.
--	**Total**	--	--	**$13,990,800**	**45,656**	

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