WATER SYSTEM CAPACITY ANALYSIS
FOR THE
SOUTH SIDE WATER SUPPLY AREA

Clarendon Hills, IL

April 29, 2015

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CHAPTER 1 : EXECUTIVE SUMMARY

STUDY GOALS AND OBJECTIVES

This study focuses on the southern portion of Clarendon Hills, located south of the Metra BNSF Railway line. For the purposes of this report, this study area has been termed the South Side Water Supply Area (SSWSA). The SSWSA is approximately 1.08 square miles consisting of both incorporated and unincorporated areas. The Departments of Community Development and Public Works are responsibly taking initial steps to plan for water supply infrastructure in an area that faces physical and logistical challenges. The identified goal of this project is to provide the Village with a decision making tool, to be used for broad brush budgeting and planning, and to guide more detailed analysis as the planning process progresses.

This study analyzes water capacity for the existing, 2015 condition, and also looks at future development in the SSWSA. This future development area is defined as the unincorporated properties south of the Village's current southern boundary down to 59th Street.

SUMMARY OF EXISTING CONDITIONS

Elevations range from 713 to 770 across the Village. The high point in the Village is located along Western Avenue, south of 55th Street, while the lower points in the Village generally follow the east village limit at Route 83.

The major land-uses of the SSWSA are residential (70%) and public right of way (20%) with smaller concentrated sections of commercial and retail focused in the Village Center and along 55th Street. The current Village population is approximately 8,600 people.

Existing Village average day water demand is estimated at 0.778 MGD, with a seasonal peaking factor of 1.5, the majority of which are residential users.

The water system infrastructure consists of approximately 29 miles of water main (42% in the SSWSA), 2 elevated and 1 ground storage reservoirs totalling 1.25 MG, and 1 pump station. The Village receives water from the City of Chicago via the DuPage Water Commission at two transfer stations, one on Middaugh Road in the north half of the Village and one on Ann Street in the south half.

System pressures generally range between 30 – 50 psi, with occasional pressure dips below 30 psi along Western Avenue, south of 55th Street. The system relies heavily upon pressures provided by the DuPage Water Commission at the transfer stations, where the delivery pressure of 90 psi is stepped down to 45 psi.

SUMMARY OF FUTURE CONDITIONS

Future conditions assume the annexation of approximately 0.23 square miles of unincorporated residential areas south of 55th street, with an estimated population increase of 884 people. The average day water demand is estimated to increase by 10% to 0.854 MGD.

SYSTEM ANALYSIS

The water system was modelled in WaterCad utilizing data provided by the Village and DuPage County. Analysis was performed for average day and peak day scenarios in the current and future condition, testing the capacity of source feed, reservoirs, and the Ann Street pump station. Two shortfalls were identified in the system.

First, the emergency storage volume was found to be limited. It is estimated that the Village should have approximately 1.7 MG of water storage compared to the existing 1.25 MG, or a shortfall of 0.5 MG. Furthermore, the operational settings of the existing reservoirs are such that the system pressures fall below 20 psi before the full storage volume can be utilized.

Second, two locations were identified as having pressure sensitivity in the Village. The first location is on the North side of the BNSF railroad, bounded by Chicago Avenue on the north, Indian Drive on the East, Burlington Avenue on the south, and Hiawatha Drive on the east. This area has old water mains in poor condition, requiring pressures to remain below 45 psi. The second pressure sensitive location is the low pressure zone adjacent to Western Avenue, south of 55th Street. Due to elevations, system pressures in this area were shown to fall below 30 psi during average and peak demands.

ALTERNATIVE IMPROVEMENTS

Alternative improvements were reviewed based on the development area and demands for the future condition. The goal of each alternative was to meet the following criteria:

1. Provide water storage equal to 2-days of average demand in the future condition.
2. Provide a minimum system pressure of 30 psi at all times during the 24 hour future peak demand simulation (when water is being delivered by the DuPage Water Commission).
3. Provide a minimum system pressure of 20 psi at all times during a 36 hour future average day emergency simulation (when water has been cut off by the DuPage Water Commission).
4. Limit pressures north of the BNSF Railroad to 50 psi.
5. Extend water main to all future development properties to the southern limits of the SSWSA.

Three alternatives were reviewed.

<u>Alternative 1: Local Pressure Zone Control (Zone Booster)</u>

This alternative creates an isolated pressure zone for the area at and surrounding Western Avenue, south of 55th street. This pressure zone is approximately 0.09 square miles, bound by 55th Street on the north, Bentley Avenue on the east, 58th Street on the south, and Richmond Avenue extended on the west. Required improvements include the construction of a 0.5 MG Elevated Storage Tank, a small booster pump station, control upgrades, and 7 miles of water main.

Alternative 2: Local Pressure Zone Control (Ann Booster)

Similar to Alternative 1, Alternative 2 creates an isolated pressure zone for the area at and surrounding Western Avenue, south of 55th street. Rather than a booster station inside the low pressure zone, Alternative 2 relies on the Ann Street Pump Station to provide higher pressures to fill the new elevated reservoir in the low pressure zone. The required improvements include the construction of a 0.5 MG Elevated Storage Tank, improvements to the Ann Street Booster Station, control upgrades, and 7 miles of water main.

Alternative 3: Western Avenue System and Ann Street Storage

Unlike Alternatives 1 and 2, Alternative 3 proposes to create a separate water network specifically for the Western Avenue low pressure area, with a separate storage supplement at the Ann Street station for the greater network. Alternatives 1 and 2 proposed an elevated reservoir that had the dual purpose of serving the Western Avenue low pressure zone, while also offering additional storage for the entire system with a wider elevation operating range. Alternative 3 allows the existing system to operate in a similar fashion as it does currently, but also adds a separate smaller system in the Western Avenue Low Pressure Zone with its own set of operating conditions. The required improvements include the construction of a 70,000 Gallon Ground Reservoir and small booster pump station in the Western Avenue low pressure zone, 0.4 MG Ground Reservoir at the Ann Street Pump Station, control upgrades, and 7 miles of water main.

DECISION MATRIX OF IMPROVEMENTS

A decision matrix has been created that assigns a score to each alternative based upon a list of criteria. Major factors have been identified that will impact the success of the future water capacity infrastructure. These factors are labelled across the top of the matrix and given a weighting percentage in accordance with defined criteria. Based on the matrix value assignments, Alternative 1 ranks the highest. This tool should be reviewed and modified by the Village as needed based upon changing priorities as development planning process progresses.

Decision Matrix of Alternatives

Alternative (Weight %)	Perform. & Operations (30%)	Capital Cost (30%)	Phasing (20%)	O&M (10%)	Gov / Leg (10%)	Score	Rank
Alternative 1	8	5	5	4	5	**5.8**	**1**
Alternative 2	4	5	5	7	5	**4.9**	**3**
Alternative 3	5	6	7	4	5	**5.6**	**2**

CHAPTER 2 : EXISTING CONDITIONS

2.1 STUDY AREA

2.1.1 Location

The Village of Clarendon Hills is located in DuPage County, 3.3 miles south of Interstate 88, 2.4 miles west of Interstate 294, and 4 miles north of Interstate 55. Clarendon Hills is bordered by Westmont on the north and west, Hinsdale and unincorporated areas to the east, and Willowbrook and unincorporated areas to the south.

The focus of this study is the southern portion of Clarendon Hills, located south of the Metra BNSF Railway line. For the purposes of this report, this study area will be termed the South Side Water Supply Area (SSWSA). The SSWSA is approximately 1.08 square miles in area consisting of both incorporated and unincorporated areas.

This study will analyze water capacity for the existing, 2015 condition, and will also look at future development in the SSWSA. This future development area is defined as the unincorporated properties south of the Village south limits down to 59th Street. See **Figure 2.1** below for a definition of the described areas.

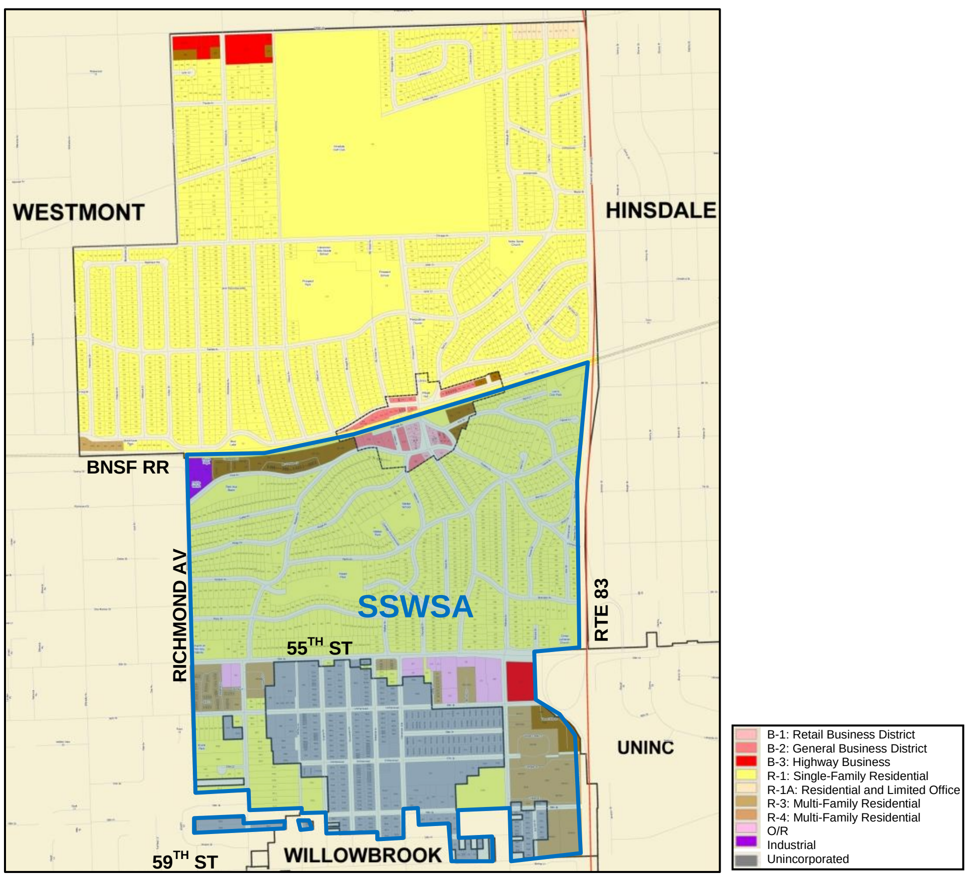


Figure 2.1: Project Study Area

2.1.2 Topography

Village topography was determined using spot elevation and contour data made available by DuPage County. These elevations are based on the North American Vertical Datum of 1988 (NAVD88) and range from 713 to 770 across the Village. The high point in the Village is located along Western Avenue, south of 55th Street, while the lower points in the Village generally follow the east village limit at Route 83.

It should be noted that the level of detail north of the BNSF is shown in less detail due to this Study's focus on the SSWSA. In general, all data on the north side of the Village was generalized in order to provide a calibrated water system model. Therefore, less elevation points were used on the north, resulting in broader contour detail.

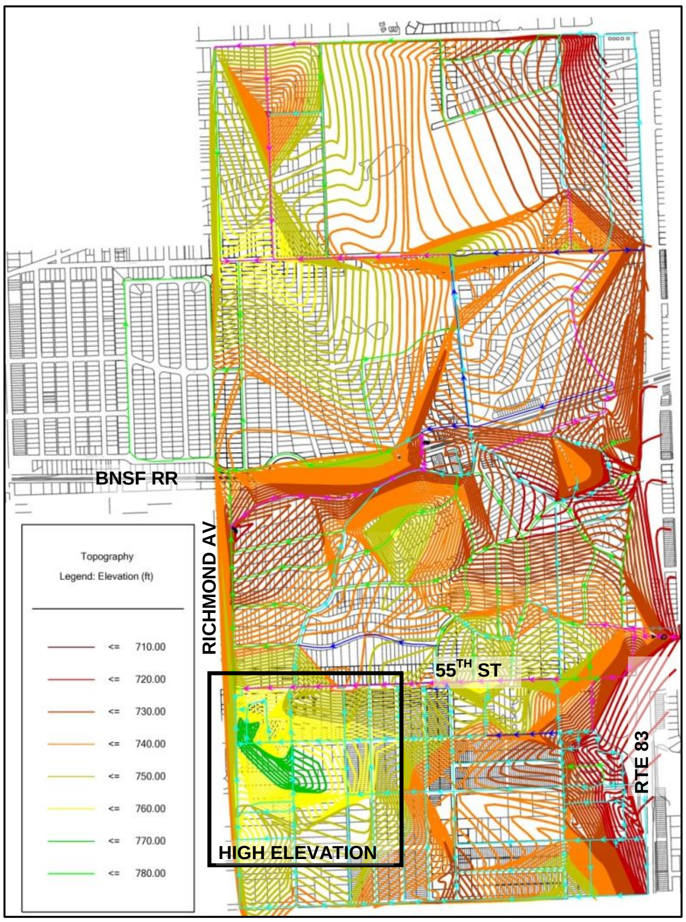


Figure 2.2: Village Topography

As will be discussed in this report, elevation differences are a driving factor in the recommendations of future water system improvements. The 57-foot range of elevation across the Village is equivalent to approximately 25 psi of water pressure. An efficient water system will aim to provide service pressures in the 30 – 50 psi range. Higher pressures may lead to water main breaks in older systems, while lower pressures jeopardize fire flow and lead to user complaints.

2.1.3 Land Use

Land use in the SSWSA, based on the Village's current Zoning Map, is broken down by area in **Table 2.1** as follows:

Table 2.1: SSWSA Land Use

Land Use	Area (mi^2)	Area%
Right of Way	0.22	20.3%
General Business District	0.007	0.6%
Highway Business	0.006	0.6%
Single-Family Residential	0.49	45.3%
Multi-Family Residential	0.10	9.2%
Office and Retail	0.024	2.2%
Industrial	0.005	0.5%
Unincorporated	0.23	21.3%
TOTAL	**1.08**	**100%**

Based on the table and zoning map and excluding Right of Way, the primary land use is for Single-Family Residential properties, followed by Multi-Family Residential Properties.

2.1.4 Population

Population was reviewed from a variety of sources in order to determine the most accurate representation in the water system model. These include the Village website, 2013 U.S. Census Bureau, and the Village's Annual Water Use Audit Forms (LMO-2) from 2012 – 2014.

Table 2.2: Village Population Summary

Source	Population
Village Website	8,500
2013 US Census Bureau	8,634
LMO-2 Audit Forms	8,486

An independent calculation was completed utilizing Village GIS parcel mapping. All parcels in Single-Family and Multi-Family Residential were grouped and summed. Multi-Family properties were broken down into an estimated number of units based upon a Google Earth street-view of the properties. The number of people per household in Single-Family zoning was estimated at 3.12 based upon the population and household counts on the Village's LMO-2 forms. The number of people per unit in Multi-Family zoning was estimated at 2 people per unit.

An initial calculation resulted in an over-estimation of the population. The residential areas were reviewed again based on a community drive-through and Google-Earth street-view. It became apparent that the Village is in a phase of residential re-construction. In some cases, multiple lots are consolidated into a single larger lot. A 10% reduction was applied to the total number of single family residential lots to account for this. The resulting final population calculation is summarized in **Table 2.3**. It should be noted that the South total excludes unincorporated areas.

Table 2.3: Village Population Calculation

Location	Population
North	4,606
South	4,135
Total	**8,741**

This slightly more conservative value was adopted for water system analysis purposes for the present day assessment.

2.1.5 Other Users

Other water users match up closely with the Land Use categories summarized in **Table 2.1**. Some specific institutions were broken out separately in order to provide a more accurate water demand. **Table 2.4** lists all non-residential water users defined in the water system analysis.

Table 2.4: Non-Residential Users

Non-Residential Users
Church
General Business District
Highway Business
Industrial (Police Dep & Public Works)
Library
Office and Retail
Residential and Limited Office
Retail & General Business District
Retail Business District
School

2.2 WATER DEMANDS

As with the population estimation, water demands were reviewed from multiple available sources, and calculated independently to corroborate available data and for entry into the water system analysis. The approach is discussed in **Sections 2.2.1 – 2.2.7** below.

2.2.1 Allocation

The Village of Clarendon Hills and the DuPage Water Commission entered into a Water Purchase and Sale Contract on June 11, 1986. This contract defined the requirements of both the Customer (Clarendon Hills and other Charter Customers) and the Commission, including the initial water allocation. Section 24 of this contract sets the Village of Clarendon Hills allocation at 0.749 MGD. No subsequent contracts were provided by the Village and it is therefore assumed that this represents the current allocation between the Village and the Commission.

2.2.2 LMO-2 Forms

The Village is required to submit an Annual Water Use Audit Form (LMO-2) to the Chicago office of the Illinois Department of Natural Resources. The LMO-2 form details the amount of water used, sold and lost by the Village within a water year (October - September). IDNR uses these forms to track individual user's compliance with the conditions of their allocation permits and to produce reports. The forms also provide the Village with a snapshot of their water system condition, where a high percentage of unaccounted for flow alerts to poor pipe condition or major leaks. **Table 2.5** provides a 3-year summary of Gross Annual Pumpage (used to estimate water delivered to the system), Accounted For Flow (used to estimate actual metered user demand plus hydrant flows), and Real Unaccounted For Flow.

Table 2.5: LMO-2 Water Use Audit Summary

Year	Gross Annual Pumpage (MGD)	Accounted For Flow (MGD)	Real Unaccounted For Flow (MGD)
2012	0.749	0.699	0.050 (6.67%)
2013	0.725	0.649	0.076 (10.5%)
2014	0.675	0.605	0.070 (10.4%)

The important value for the purposes of the system analysis is the Gross Annual Pumpage. This value represents the actual Volume entering the Village system. This data is presented graphically in **Figure 3** below. It is evident that there is a trend of decreasing water usage over the past 3 years.

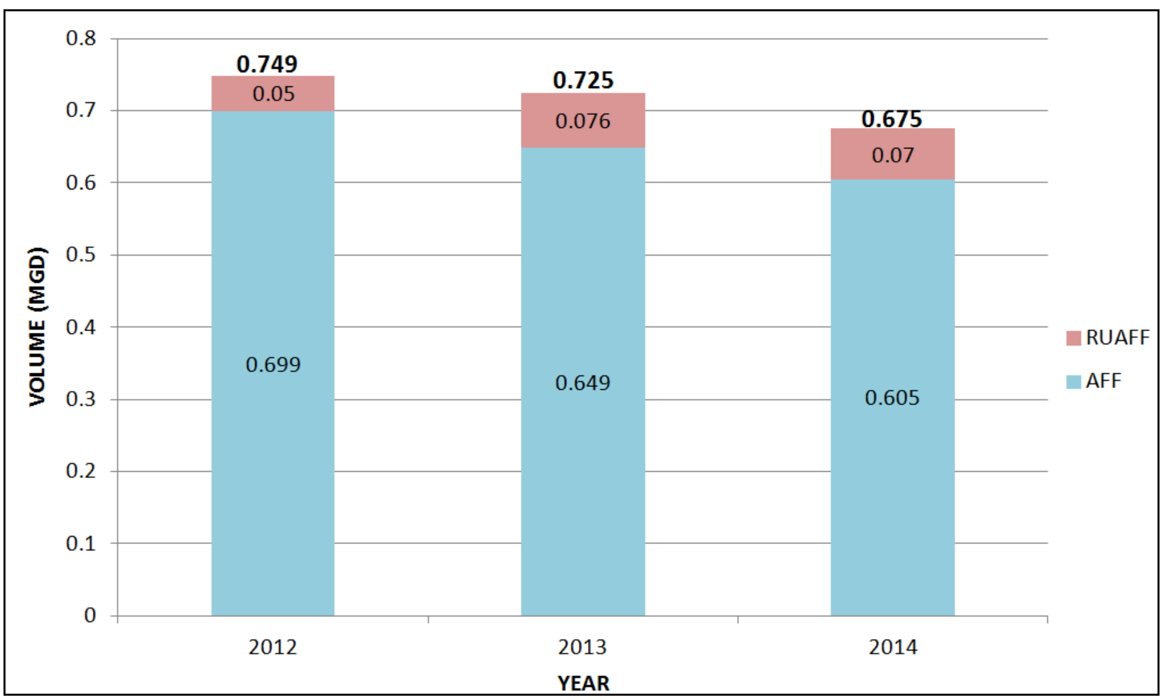


Figure 2.3: LMO-2 Water Use Audit Summary

The similarity between the Gross Annual Pumpage values on the LMO-2 forms and the 0.749 MGD water allocation by contract, suggest that the current allocation is realistic for the present day.

2.2.3 Meter Readings

The Village provided daily metering summaries for 2014 for analysis. These summary sheets provide individual meter readings from both the Ann and Maple transfer stations. The combined daily values total the daily volume of supply coming into the Village from the DuPage Water Commission on a given day. **Figure 2.4** provides a graph of these daily totals, together with the minimum, maximum, and average daily water usage.

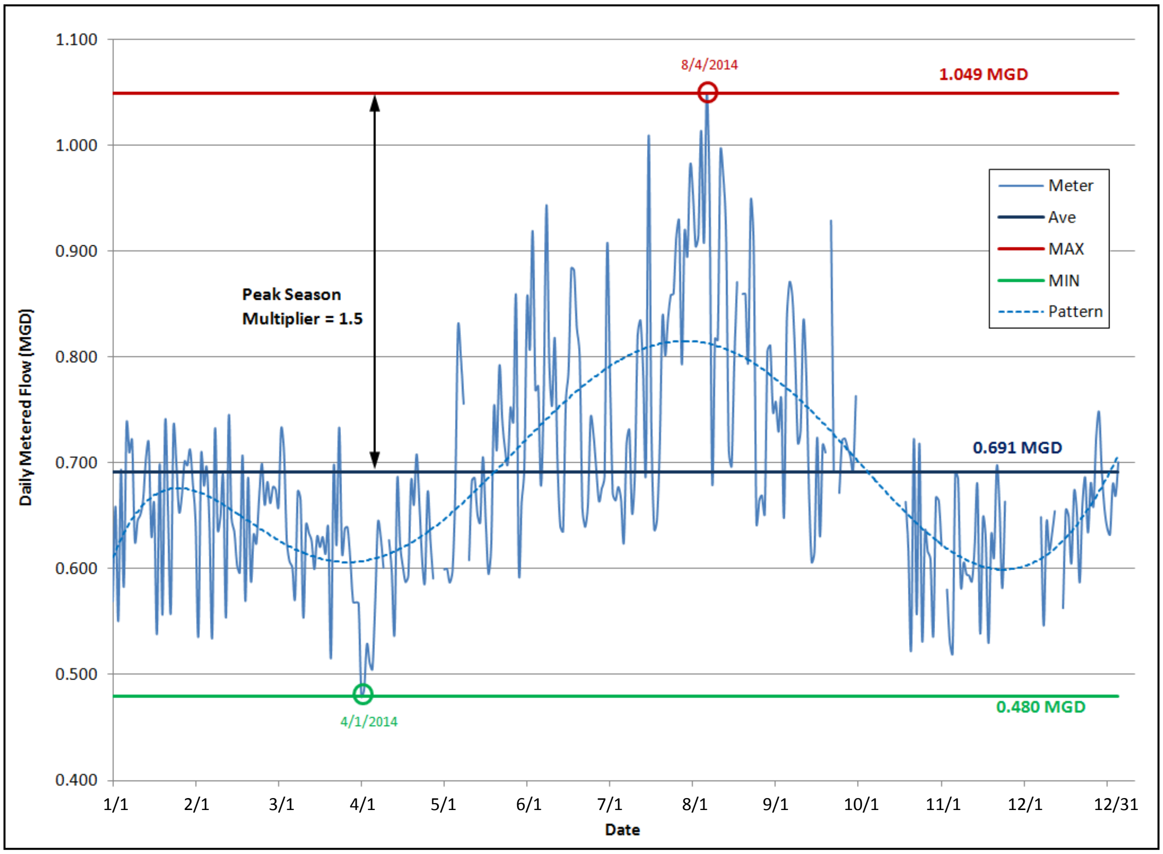


Figure 2.4: 2014 Daily Metered Flow

Flows for 317 days (48 days of data were either missing or un-readable) were averaged for a value of 0.691 MGD. A minimum flow of 0.480 MGD was taken on April 1, 2014. A maximum flow of 1.049 MGD was taken on August 4, 2014. Based on these values, a seasonal peaking factor of 1.5 was adopted for the water system analysis.

2.2.4 Calculated Demand

Demand was calculated independently of allocation, LMO-2 reporting, and metered flows. This calculation builds upon the population and non-residential user estimates discussed in **Sections 2.1.3 – 2.1.5**. Daily water use rates were then defined for each type of user.

Water use rates for residential customers were found to be the primary driving factor for an accurate demand calculation. After an initial rate of 100 gal/person/day [gppd] was found to be too conservative, the LMO-2 forms were reviewed to define a more accurate rate. Rates of 76, 71, and 66 gppd were calculated for 2012, 2013, and 2014 respectively. In order to simplify the model, hydrant flows and unaccounted for flow were also added into the residential daily rate. This increased the rates to 83, 81, and 75 gppd. Conservatively, a rate of 85 gppd was adopted for residential users.

All other user rates were adopted based on typical values defined in *Water Distribution Systems Handbook* (Mays, 2000). The final demand calculation is summarized in **Table 2.6** below.

Table 2.6: Existing Conditions Average Day Calculated Demand

DEFINITION		SSWSA		NORTH		VILLAGE	
Type	**Rate (gppd)**	**# Users**	**Total (GPD)**	**# Users**	**Total (GPD)**	**# Users**	**Total (GPD)**
Church	5	800	4,000	400	2,000	1,200	6,000
General Business District	15	100	1,500	210	3,150	310	4,650
Highway Business	30	50	1,500	300	9,000	350	10,500
Library	15	-	-	200	3,000	200	3,000
Industrial (Police Dep & Public Works)	30	70	2,100	-	-	70	2,100
Multi-Family Residential	85	1,380	117,300	224	19,040	1,604	136,340
Office & Retail	15	200	3,000	-	-	200	3,000
Retail & General Business District	15	20	300	-	-	20	300
Retail Business District	15	150	2,250	-	-	150	2,250
Residential and Limited Office	30	-	-	360	10,800	360	10,800
School	15	345	5,175	1,224	18,360	1,569	23,535
Single-Family Residential	85	2,755	234,175	4,022	341,870	6,777	576,045
TOTAL			**371,300**		**407,220**		**778,520**

2.2.5 Adopted Demand

Water demands from the various sources and the calculated demand are compared in **Table 2.7** below.

Table 2.7: Summary of Existing Demands by Source

Source	Daily Demand (MGD)
Contract Allocation	0.749
2012 LMO-2	0.749
2013 LMO-2	0.725
2014 LMO-2	0.675
2014 Meter Readings (Ave)	0.691
Calculated	0.778

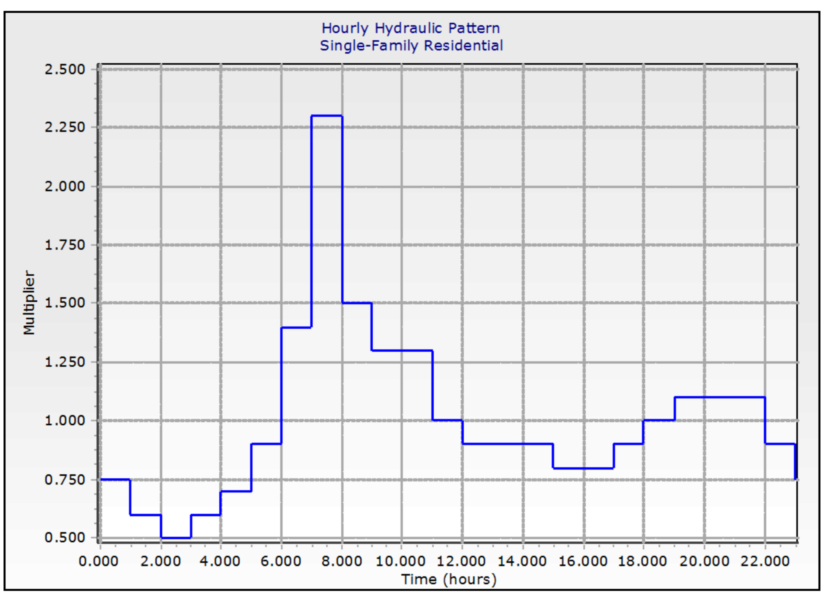
For the purposes of the water system analysis, the conservative calculated daily demand of 0.778 has been adopted.

2.2.6 Diurnal Curves

Diurnal curves have been applied to the various users in order to most accurately represent water demands during specific periods of the day. Rather than a static model, this type of approach provides insight into tank level trends and critical periods when pressure conditions drop below desired levels. All curves are based upon weekday patterns.

2.2.6.1 Residential

Residential water use typically peaks in the mornings, with a secondary lesser peak in the evenings. Curves for similar types of user areas were reviewed, resulting in the following adopted patterns shown in **Figure 2.5** below. It should be noted that the start time is midnight, with subsequent hours provided in military time.

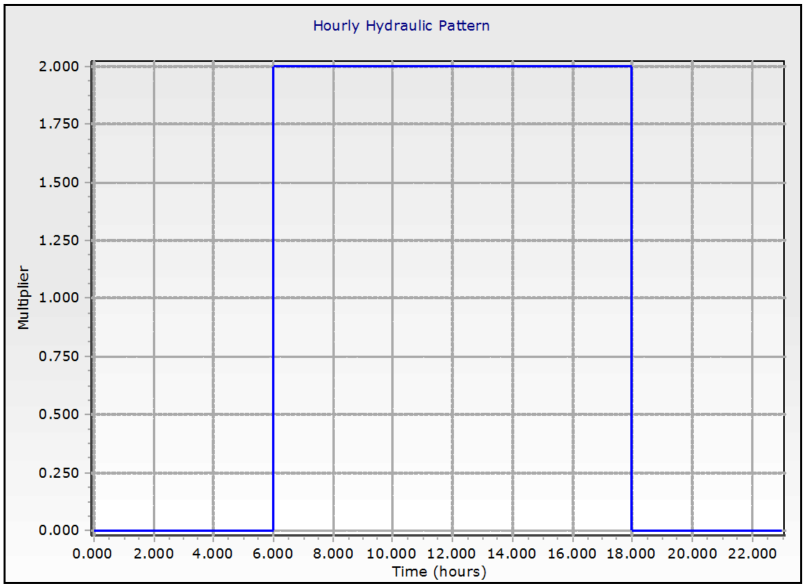


Time from Start (hours)	Multiplier
1.000	0.600
2.000	0.500
3.000	0.600
4.000	0.700
5.000	0.900
6.000	1.400
7.000	2.300
8.000	1.500
9.000	1.300
10.000	1.300
11.000	1.000
12.000	0.900
13.000	0.900
14.000	0.900
15.000	0.800
16.000	0.800
17.000	0.900
18.000	1.000
19.000	1.100
20.000	1.100
21.000	1.100
22.000	0.900
23.000	0.750

Figure 2.5: Residential Diurnal Curve

2.2.6.2 Non-Residential

Non-residential water use is a relatively minor factor in the Village of Clarendon Hills. This is primarily due to the lack of industrial and other high-water users. Because of this, all non-residential curves assume a constant use between the hours of 6 a.m. – 6 p.m. as shown in **Figure 2.6** below.



Time from Start (hours)	Multiplier
1.000	0.000
2.000	0.000
3.000	0.000
4.000	0.000
5.000	0.000
6.000	2.000
7.000	2.000
8.000	2.000
9.000	2.000
10.000	2.000
11.000	2.000
12.000	2.000
13.000	2.000
14.000	2.000
15.000	2.000
16.000	2.000
17.000	2.000
18.000	0.000
19.000	0.000
20.000	0.000
21.000	0.000
22.000	0.000
23.000	0.000

Figure 2.6: Non-Residential Diurnal Curve

2.2.7 Peak Factors

Based on a review of the meter readings in 2014, a seasonal peak factor of 1.5 has been used in the system analysis (see **Section 2.2.3** for discussion). This seasonal peak factor is applied to review peak demand scenarios in both the existing and future conditions.

A daily peak factor of 2.3 has been defined in the residential diurnal curve, timed to occur at 7:00 a.m. (see **Section 2.2.6.1** for discussion). Technically, there is no peak factor assumed for non-residential flow. However, a multiplier of 2 is shown in Figure 6 such that all flow on a given day takes place in the 12 hour period between 6 a.m. and 6 p.m.

2.3 WATER SYSTEM COMPONENTS

2.3.1 Water Main

A digital copy of the Village's water main atlas was provided during initial coordination meetings. The Village's water operation consultant, M.E. Simpson, indicated that the atlas was up to date for the SSWSA, but that the north side of the Village still required updates. A comprehensive list of water main locations and condition ratings was also available on the Village's website. Based on these sources, it is estimated that the total length of water main is 29.11 miles, ranging in diameter from 4" – 12". Based upon the atlas review, and building of the water model, the SSWSA infrastructure is summarized in **Table 2.8** below.

Table 2.8: SSWSA Water Main Summary

Dia. (in)	Village Total (mi)	SSWSA Total (mi)	SSWSA %
4	0.66	-	-
6	14.37	4.92	34.2%
8	8.50	4.75	55.9%
10	1.58	0.57	36.1%
12	4.00	2.11	52.8%
Total	**29.11**	**12.35**	**42.4%**

According to the Village's Water Main Inventory Matrix, the average water main age is approximately 55 years and the average condition rating is 17 (out of a maximum 20).

2.3.2 Reservoirs

The Village utilizes 2 elevated storage reservoirs and 1 ground storage reservoir, as summarized in **Table 2.9** below.

Table 2.9: Reservoir Summary

Reservoir	Type	Year Built	Design Volume (MG)	Current Operating Volume
Ann Street (East)	Ground	Unavailable	0.500	0.370
Burlington Ave (North)	Elevated	1954	0.250	0.082
Park Ave (South)	Elevated	1971	0.500	0.128
Total			**1.25**	**0.580**

The current operating volumes were calculated based upon the SCADA set-points for the elevated tanks, and the pump capabilities at the Ann Street Ground Reservoir.

The SCADA screens provide set-points that are based upon the water level in the Burlington Avenue Tank, which is also tied hydraulically to the Park Avenue Tank. The system keys the Ann Street pump station to turn on when the water level in the Burlington Avenue Tank is at 18-feet, and turn off when the level is at 22.5-feet. Furthermore, the system keys the Maple and Ann Street transfer stations to stop flow into the system when the Burlington tank level is at 27.0 –feet. The overall operating range of the elevated tanks can therefore be calculated as 9 feet (27.0 – 18.0).

From the standpoint of the Ann Street Ground Reservoir, Village operators indicated that the pumps cannot lift the final 4-feet of water.

These levels have been illustrated in **Figure 2.7.**

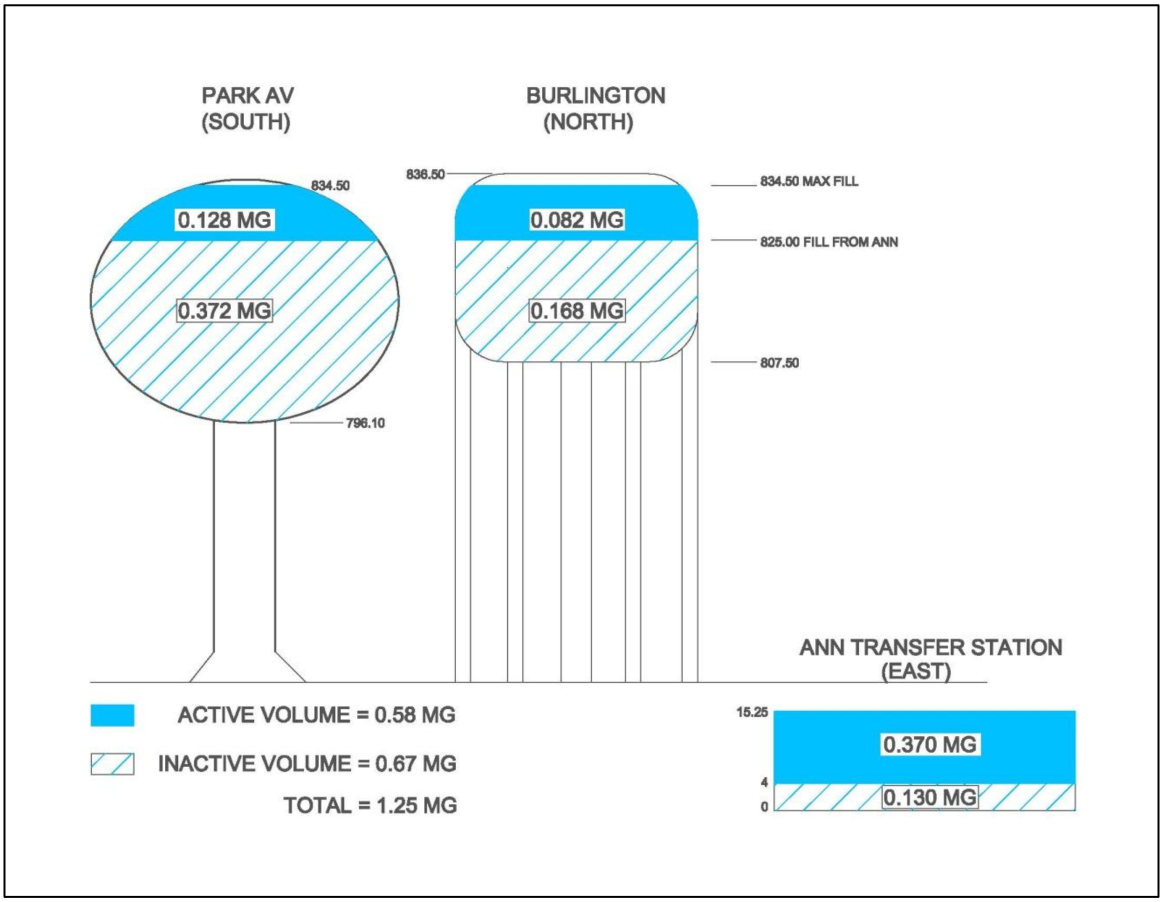


Figure 2.7: Reservoir Levels

2.3.3 Transfer Stations

The Village of Clarendon Hills receives water from the City of Chicago via the DuPage Water Commission at two transfer stations. The north transfer station is located on Middaugh Road between Walnut Street and Chicago Avenue. The south transfer station is located on Ann Street, just north of Sheridan Road.

Photo 2.1: Middaugh Rd. Transfer Sta.

Photo 2.2: Ann Street Transfer Sta.

With the exception of the process piping, the stations are almost identical in design. The system supply line enters the metering station where the Commission records the volume of delivered water. Supply then enters a second, adjacent building, operated by the Village, where the pressure from the Commission is stepped down. Flow is metered a second time in this building prior to entering the Village's distribution system, or in the case of the Ann Street Station, flow can be diverted to the underground storage reservoir.

Flow set points are defined to regulate the flow into the system or reservoir and pressure reducing valves are used to step down the pressure. A summary of the set points and 2014 annual flows are summarized in **Table 2.10** below.

Table 2.10: Transfer Stations Summary

Setpoint / Measure	Ann Street	Middaugh Road
Incoming Commission Pressure (psi)	75-90	75-90
Outgoing System Pressure (psi)	46	52
Outgoing System Flow Rate (GPM)	238	189
Ground Reservoir Fill Rate (GPM)	350	-
2014 Metered System Flow (MG)	123.5	108
2014 Metered Reservoir Fill (MG)	18.5	-

2.3.4 Ann Street Pump Station

A pump station is located at the Ann Street Transfer and Ground Reservoir Site. These pumps are used to lift water from the reservoir to feed the system and fill the elevated reservoirs at Burlington Avenue and Park Avenue. There are 3 identical pumps in the pump station, two of which operate as alternating duty / standby modes. The third pump may be used in emergencies but is rarely utilized. Flow tests were performed on the pumps in 2013, resulting in the duty points shown in **Table 2.11** below.

Table 2.11: Ann Street Pump Measured Duty Points

Measure	Pump 1	Pump 2
Point 1	370 gpm @ 64 psi	370 gpm @ 62 psi
Point 2	600 gpm @ 54 psi	620 gpm @ 52 psi
Shutoff Pressure	70 psi	74 psi

Village operating staff indicated the pump station delivers a flow of 650 gpm. For analysis purposes, the duty point was therefore adopted at 650 gpm at 55 psi of pressure.

2.4 WATER SYSTEM OPERATION

2.4.1 Description

The Village system is driven by maintaining water level set-points in the two elevated reservoirs. The specific set point values are based on the Burlington (North) reservoir, but the two are hydraulically connected and will therefore have relatively the same water elevations at a given time. The Controlling Tower Elevation is roughly 830.1 (equal to a level of 22.6 in the Burlington Reservoir). Assuming this Controlling Tower Elevation as the current water level, the system will operate as summarized in **Table 2.12**.

Table 2.12: System Operation

Level	Elevation	Tank Level Movement	Ann Flow	Middaugh Flow	Ann Pump 1	Ann Pump 2
22.6	830.1	↓↓↓↓ Empty	Yes	Yes	No	No
18.0	825.5	↓↓↓↓ Empty	Yes	Yes	Yes	No
15.0	822.5	↓↓↓↓ Empty	Yes	Yes	Yes	Yes
18.0	825.5	↑ Fill	Yes	Yes	Yes	No
22.5	830.0	↑ Fill	Yes	Yes	No	No
25.5	833.0	↑ Fill	Reduced	Yes	No	No
26.5	834.0	↑ Fill	No	Yes	No	No
27.0	834.5	↓↓↓↓ Empty	No	No	No	No
26.5	834.0	↓↓↓↓ Empty	No	Yes	No	No
25.5	833.0	↓↓↓↓ Empty	Reduced	Yes	No	No
22.6	830.1	↓↓↓↓ Empty	Yes	No	No	No

Generally speaking, the valves at the transfer stations remain open at set flow rates until the water levels in the elevated reservoirs rise towards their overflow points, at which time flow from the stations are first reduced, then closed completely. On the other hand, as water levels get low in the elevated reservoirs, the pumps at Ann Street kick on to try and raise the levels back. Either 1 or 2 pumps will turn on depending on how low the water gets.

2.4.2 Pressure Modes

Based on the water system operation discussed in **Section 2.4.1**, the system can receive pressure from three sources. These sources are discussed in the following section and are used as the basis for the modeling assessment. These modes are summarized in **Table 2.13** below.

Table 2.13: Pressure Modes

Mode	Pressure Range at Source	Description
Transfer Station	46 – 52 psi	Pressure in the system is controlled by the outlet pressures at the Ann and Middaugh transfer stations after the pressure reducing valve.
Elevated Reservoir	33 – 50 psi	Pressure in the system is controlled by the vertical water column in the elevated reservoir. The pressure range represents what would be measured at the Park Street (South) Reservoir at the lowest and highest possible water levels.
Ann Street Pump Station	50 psi	Pressure in the system is controlled by the outlet of the Ann Street pump station. This pressure is assumed to be constant in the system analysis.

CHAPTER 3 : FUTURE CONDITIONS

3.1 DEVELOPMENT AREA

3.1.1 Location

The future development area is defined as the unincorporated properties south of the Village south limits down to 59th Street. This 0.23 square mile area is highlighted in **Figure 3.1** below.

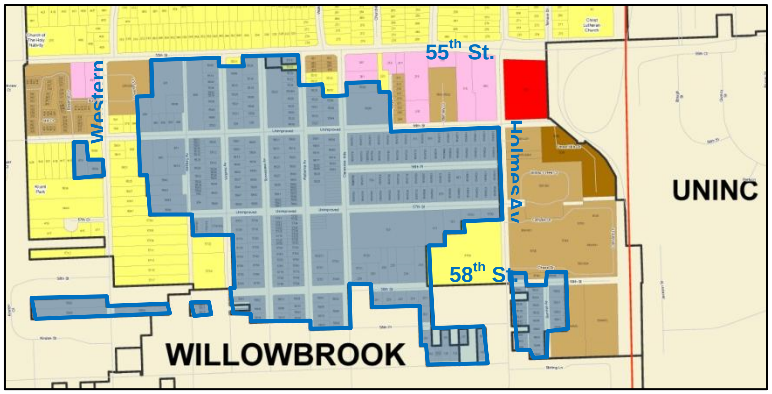


Figure 3.1: Future Development Area

3.1.2 Topography

The elevation range in the future development area ranges from 720 to 770, generally sloping down from west to east. Portions of this future area are located in the higher elevation zone as pointed out in **Figure 3.2** below.

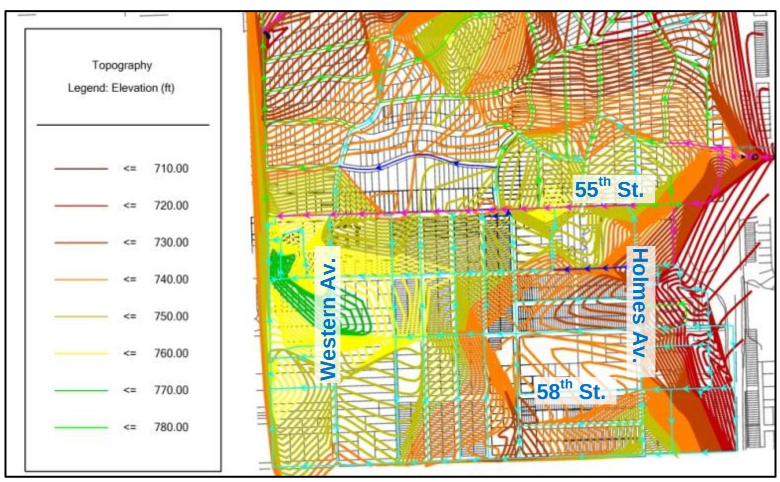


Figure 3.2: Topography

3.1.3 Land Use

The Village has indicated that the future land use of the area will retain its residential character. The following land uses have therefore been assumed for the development of the existing unincorporated areas.

Table 3.1: Future Development Area Land Use

Land Use	Area (mi^2)	Area%
Right of Way	0.038	14%
Single-Family Residential	0.217	81%
Multi-Family Residential	0.013	5%
TOTAL	**0.268**	**100%**

3.1.4 Population

Population for the future development area was based upon a count of the existing residential units from Google Earth aerials and a street view review of multi-family residential units. The number of people per household in Single-Family zoning was estimated at 3.12 based upon the population and household counts on the Village's LMO-2 forms. The number of people per unit in Multi-Family zoning was estimated at 2 people per unit.

The resulting population estimate for the future development area is summarized in **Table 3.2** below.

Table 3.2: Future Development Area Population Calculation

Type	# Units	People / Unit	Population
Single-Family Residential	191	3.12	596
Multi-Family Residential	144	2	288
Total			**884**

Based upon the summary in Section 2.1.4, the total future population of the Village is summarized in **Table 3.3** below.

Table 3.3: Future Village Population Calculation

Location	Population
SSWSA (Existing)	4,135
SSWSA (Future Development)	884
SSWSA Subtotal	5,019
North	4,606
Total	**9,625**

3.2 WATER DEMAND

Water demand calculation builds upon the population estimates discussed in **Section 3.1.4**. Water use rates were then applied to determine the total average day demand. As discussed in **Section 2.2.4**, a residential rate of 85 gppd is used which includes provisions for hydrant flows and unaccounted for flow. The final future demand calculation is summarized in **Table 3.4** below.

Table 3.4: Future Condition Average Day Calculated Demand

Location	Total (GPD)
SSWSA (Existing)	371,300
SSWSA (Future Development)	75,140
SSWSA Subtotal	446,440
North	407,200
Total	**853,660**

CHAPTER 4 : SYSTEM ANALYSIS

4.1 WATERCAD MODEL

4.1.1 General

The WaterCAD model was built using the following inputs discussed previously in the report:

- DuPage County topography data
- ME Simpson water atlases
- As-Builts for the Transfer Stations and Reservoirs
- Pump data based on flow testing
- SCADA screens review with Village water operating staff
- Adopted population and demand calculations
- Village Zoning Map
- Village development write-ups for the future SSWSA

4.1.2 Calibration

Model calibration is a critical step in assessing the accuracy of the results and the quality of the analysis. Calibration was performed in three ways. First, the demand was calibrated to correlate with the metered, and LMO-2 data. A guess and check approach was taken with the model, leading to adjustments in the population data (see **Section 2.1.4**) and the user demand rates (see **Section 2.2.4**). The adopted average day demand for the current condition is 0.778 MGD, which compares to other demand data as shown in **Table 4.1**.

Table 4.1: Comparison of Adopted Model Demand to Available Data

Source	Daily Demand (MGD)	% Difference to Adopted Demand
Contract Allocation	0.749	(3.7%)
2012 LMO-2	0.749	(3.7%)
2013 LMO-2	0.725	(6.8%)
2014 LMO-2	0.675	(13.2%)
2014 Meter Readings (Ave)	0.691	(11.2%)

Higher percent differences are represented as the years progress due to a declining pattern of water use. The adopted value of 0.778 MG remains conservative, and was refined to be within 5% of the Contract Allocation demand.

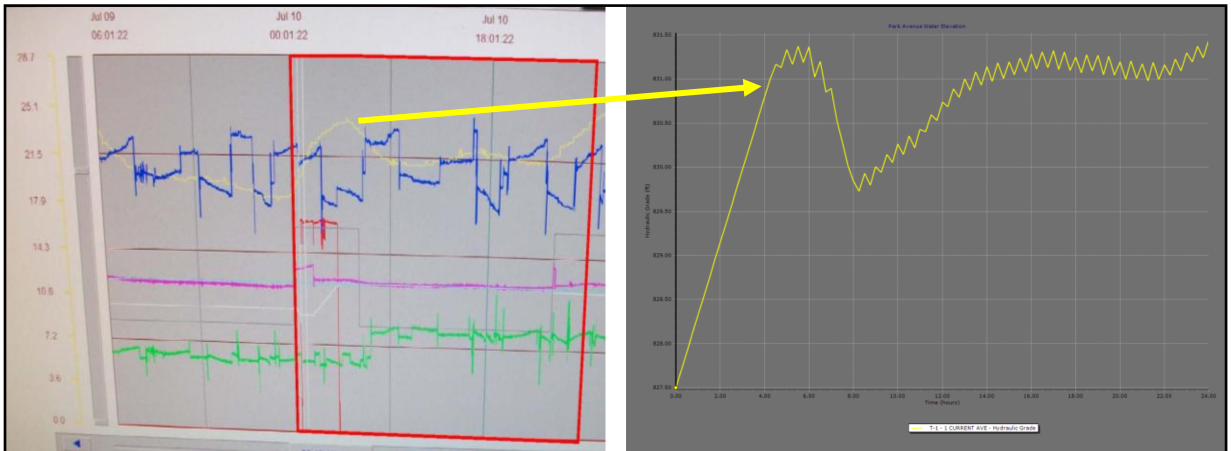
The second method of calibration was to review model pressures against static pressure readings in the system. Hydrant flow tests at 201 locations were reviewed for static pressure reading, location, date and time, and were checked against the model results at the same time in the 24-hour simulation results. Results are shown in **Figure 4.1** below.

North

South

Figure 4.1: WaterCAD Pressure Calibration

Finally, tank level patterns output by the model were compared to trends shown on the Village's Central SCADA screen. Since the SCADA data could not be extracted, a photograph was taken to capture the trend for a three day period in July of 2014. A comparison of the screen trend and the WaterCAD output for a 24-hour period are compared in **Figure 4.2** below.



SCADA Screen | Model Output

Figure 4.2: Reservoir Water Level Pattern Calibration

The exhibit shows that the model was able to replicate the elevated reservoir fill / empty pattern shown on the SCADA trending screen. The two elevated reservoirs will typically fill overnight when demand is low, then empty during the peak morning period around 6 a.m., followed by a steady recovery throughout the day until the pattern repeats itself overnight.

4.1.3 Adopted Scenarios

After model calibration, specific Scenarios were defined in order to assess the performance of the water system in the current and future condition. Three types of Scenarios were defined in order to review the system for each type of Pressure Mode defined in **Section 2.4.2**. Each of these modes were tested for performance in the current and future condition, based on average day demands as well as peak seasonal demands. The twelve Scenarios are summarized in **Table 4.2** below.

Table 4.2: Scenarios Summary

Scenario	Demand	Condition	Pressure Mode	Description
1	Average Day	Current	Transfer Station	These scenarios review the system performance when the Ann and Middaugh transfer stations are providing flow to the system. Current and future conditions are tested for system performance during Average Day and Peak Annual Day Demand.
2	Peak Day			
3	Average Day	Future		
4	Peak Day			
5	Average Day	Current	Elevated Reservoir	These scenarios review the system performance when the Elevated Reservoirs are the sole source of supply to the system. Current and future conditions are tested for system performance during Average Day and Peak Annual Day Demand.
6	Peak Day			
7	Average Day	Future		
8	Peak Day			

Scenario	Demand	Condition	Pressure Mode	Description
9	Average Day	Current	Ann Street Pump Station	These scenarios review the static system performance when the Ann Street Pump Station is operating. Current and future conditions are tested for system performance during Average Day and Peak Annual Day Demand.
10	Peak Day			
11	Average Day	Future		
12	Peak Day			

4.2 TRANSFER STATION SCENARIOS ANALYSIS

A 24-Hour Extended Period Simulation was carried out assuming system feed from the Ann Street and Middaugh Avenue transfer stations. System pressures were generally found to be acceptable, with the exception of a small low pressure zone that developed on Western Avenue, south of 55^{th}. Water fluctuation in the elevated reservoirs was minor, suggesting stable flows into the system from the transfer stations for current and future conditions.

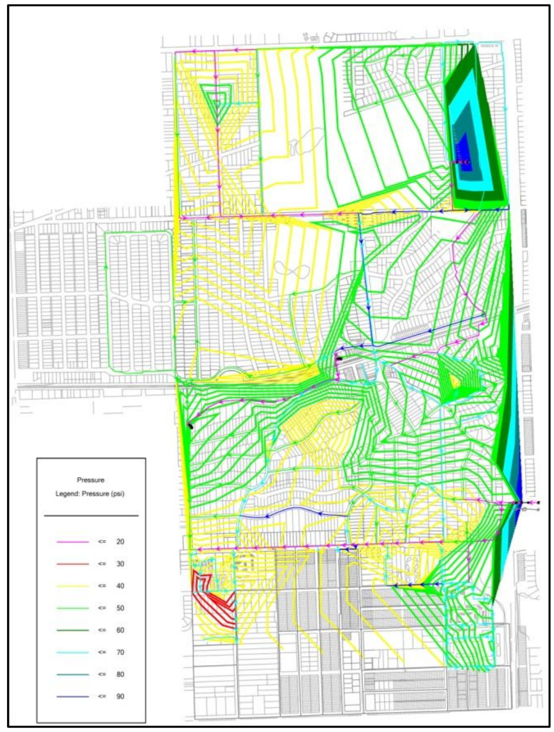
4.2.1 Pressure Results

Pressure results for the North (N), South (S), and Future Development Area (F) are summarized in **Table 4.3** below.

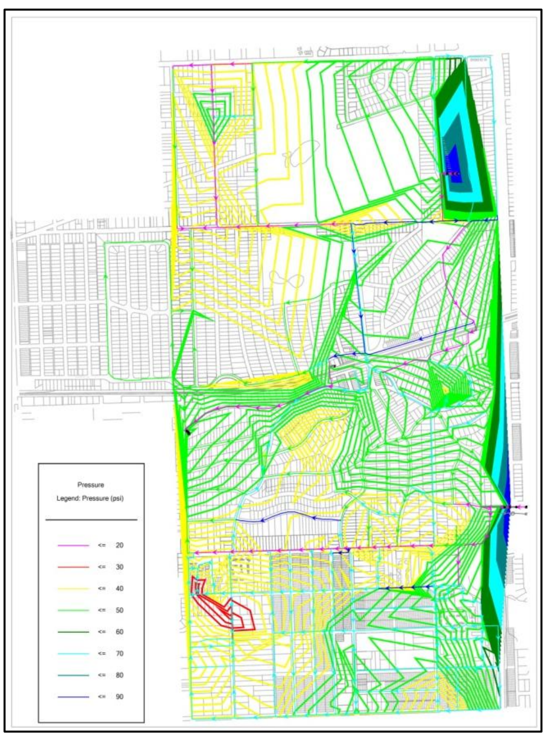
Table 4.3: Scenarios Summary

Pressure (psi)	Scenario 1 (9 a.m.)		Scenario 2 (9 a.m.)		Scenario 3 (8 a.m.)			Scenario 4 (8 a.m.)		
	N	S	N	S	N	S	F	N	S	F
Min	32	27	31	26	31	27	31	30	25	30
Ave	41	41	40	40	40	41	38	39	40	37
Max	53	50	52	50	53	50	51	52	49	50

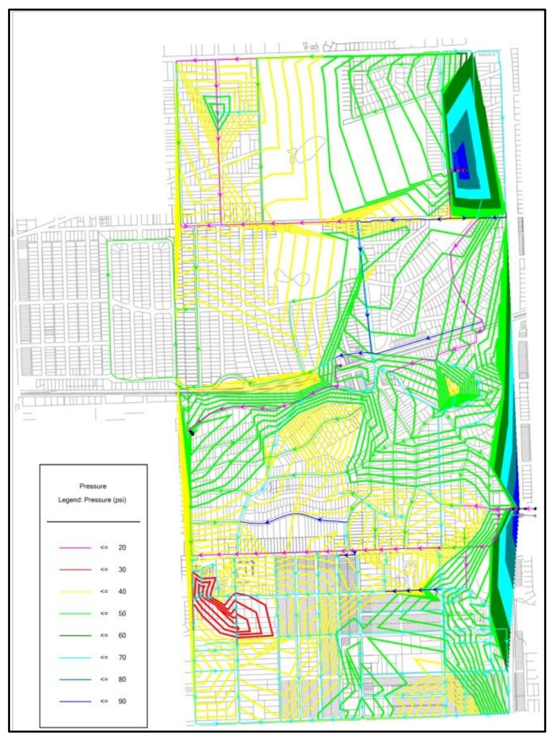
1. Current Condition – Average Day (9 A.M.)



2. Current Condition – Peak Day (9 A.M.)



3. Future Condition – Average Day (8 A.M.)



4. Future Condition – Peak Day (8 A.M.)

Figure 4.3: Pressure Results – Transfer Station Scenarios

4.2.2 Water Storage Results

Tank water levels fluctuate between 827.5 and 831.5 for both the North and South elevated storage reservoirs. This 4-foot fluctuation in the water column indicates that flow in the system is relatively stable when feed is coming from the transfer stations in both the current and future conditions. Ann Street pumps were not required to turn on (825.5) and the step-down of flows from the transfer stations was not required (833.0). Water levels in the Ann Street ground storage reservoir were not reviewed for these scenarios.

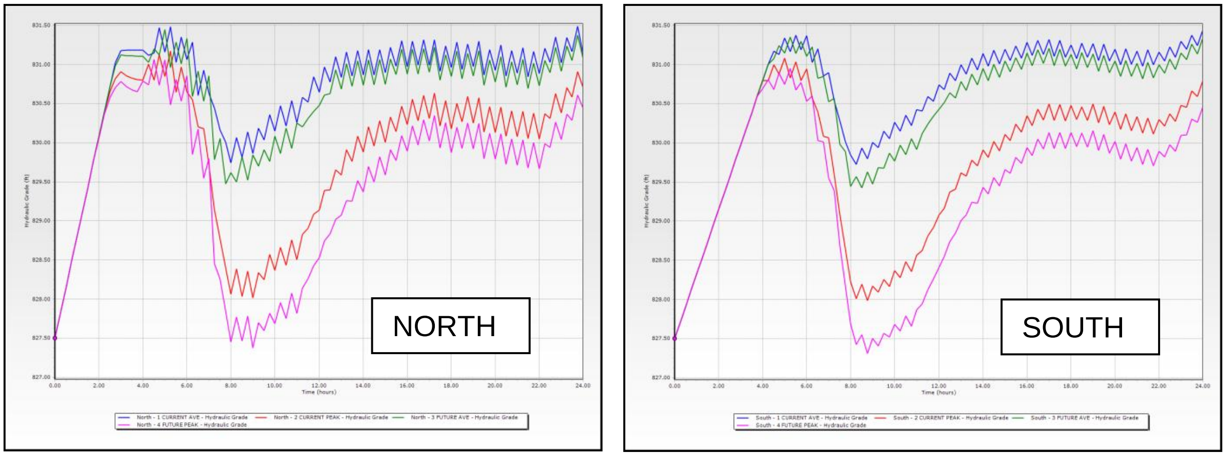


Figure 4.4: Reservoir Levels – Transfer Station Scenarios

4.3 ELEVATED STORAGE MODE – ANALYSIS

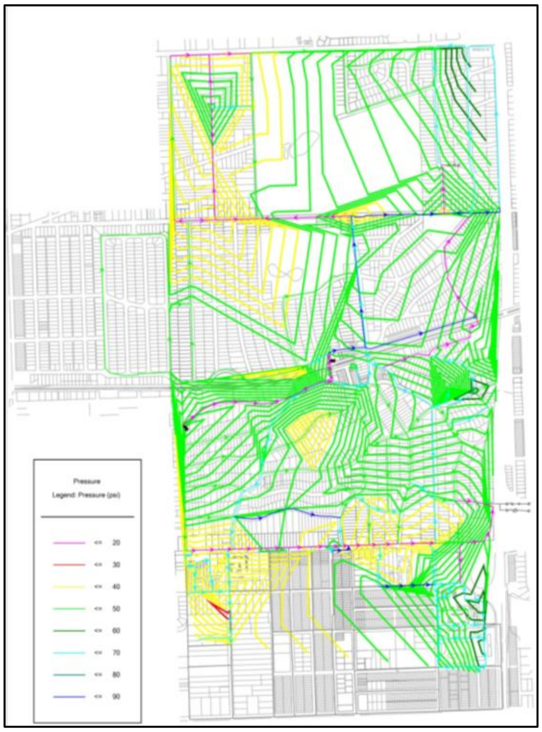
A 24-Hour Extended Period Simulation was carried out assuming system supply from the elevated reservoirs only. These scenarios are primarily used to determine the effective water levels that will provide acceptable pressures to the system. Each scenario started with full tanks at midnight. The presented results are based on pressures 1 hour into the simulation, and pressures observed when the tanks were drained to 50% full. As shown in the following sections, sizeable areas in the Village drop below 30 psi as the reservoirs drain.

4.3.1 Pressure Results

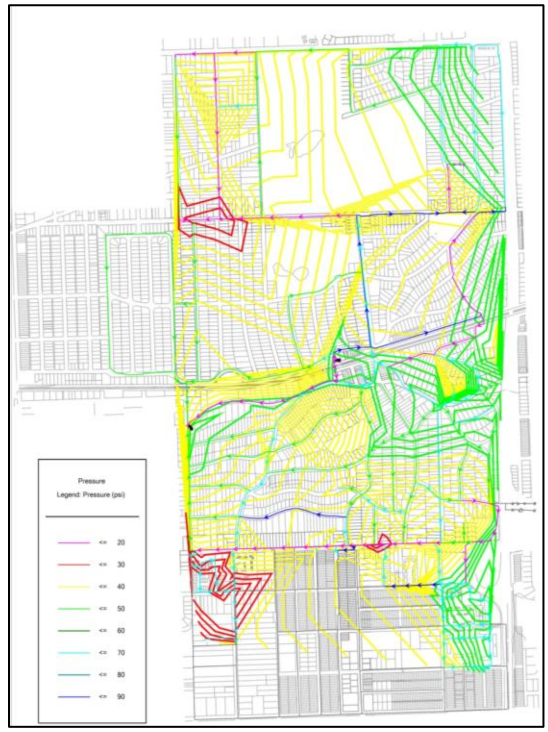
Pressure results for the North (N), South (S), and Future Development Area (F) are summarized in **Table 4.4** below.

Table 4.4: Scenarios Summary

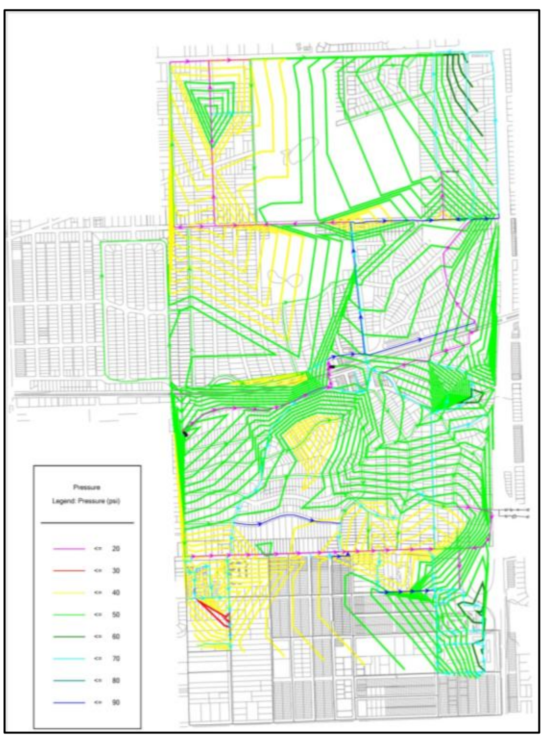
Reservoir Level	Pressure (psi)	Scenario 5		Scenario 6		Scenario 7			Scenario 8		
		N	S	N	S	N	S	F	N	S	F
Full (A)	Min	33	29	33	28	33	29	33	33	28	32
	Ave	42	43	42	43	42	43	40	42	42	40
	Max	54	52	54	52	54	52	53	54	52	53
50% (B)	Min	27	23	26	21	27	22	27	25	20	24
	Ave	36	37	35	36	36	37	34	34	35	32
	Max	48	46	47	45	48	46	47	46	44	45



5A. Current Condition – Average Day (Full) 1 A.M.



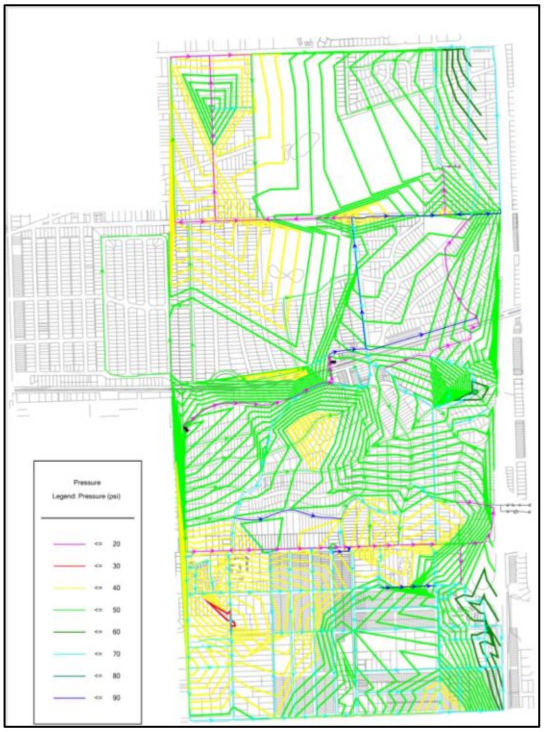
5B. Current Condition – Average Day (50% Full) 11 A.M.



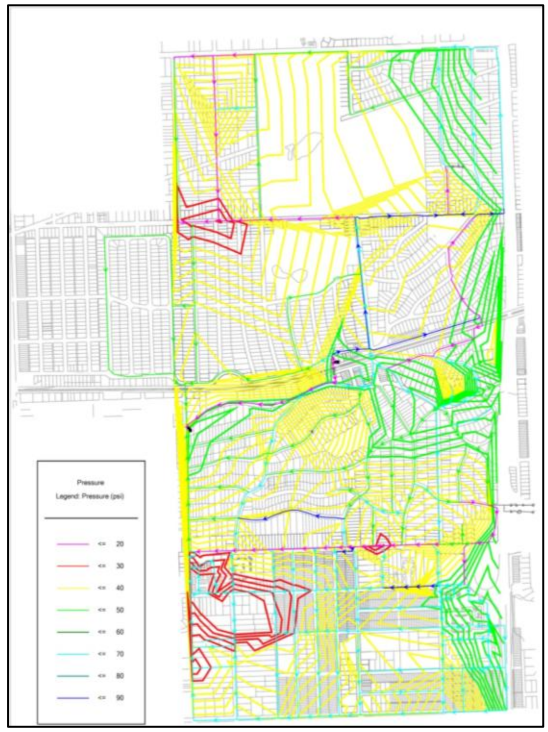
6A. Current Condition – Peak Day (Full) 1 A.M.

6B. Current Condition – Peak Day (50% Full) 8 A.M.

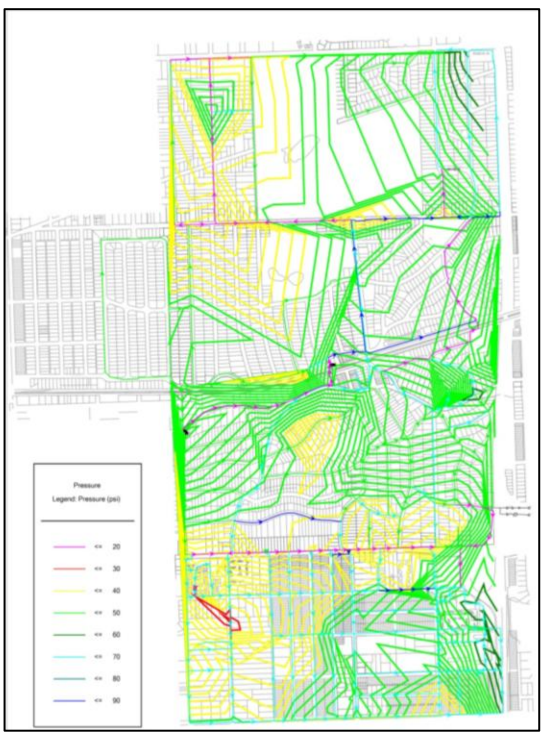
Figure 4.5: Pressure Results – Elevated Reservoir Scenarios (Current Condition)



7A. Future Condition – Average Day (Full) 1 A.M.



7B. Future Condition – Average Day (50% Full) 10 A.M.



8A. Future Condition – Peak Day (Full) 1 A.M.

8B. Future Condition – Peak Day (50% Full) 8 A.M.

Figure 4.6: Pressure Results – Elevated Reservoir Scenarios (Future Condition)

Table 4.4 and **Figures 4.5-4.6** point out once more that a low pressure zone develops along Western Avenue, south of 55th. Pressure in this area actually drops below 30 psi within 1-hour after the reservoirs begin to empty. The data also shows that a pressure problem develops in other areas of the Village as the water level drops.

These reservoirs cannot be relied on alone to keep the system above 30 psi. The transfer stations or the Ann Street pumps must be active in order to keep the system pressures above this pressure threshold.

4.4 PUMP STATION MODE – ANALYSIS

A Static Simulation was carried out assuming system feed from Ann Street Pump Station only. The purpose of this simulation is to review the system performance based upon a supply on demand approach, in which pumps would operate more often to keep system pressure. The following sections indicate that the pumps do a good job of maintaining pressure above 30 psi.

4.4.1 Pressure Results

Pressure results for the North (N), South (S), and Future Development Area (F) are summarized in **Table 4.5** below.

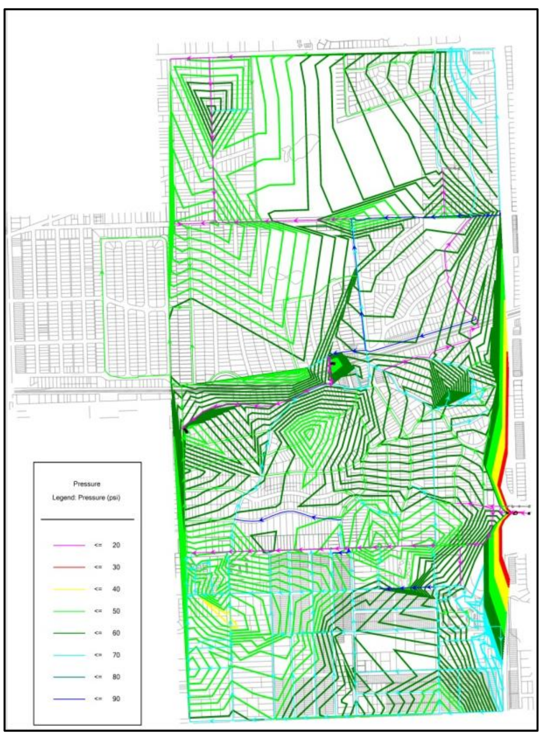
Table 4.5 Scenarios Summary

Pressure	Scenario 9		Scenario 10		Scenario 11			Scenario 12		
(psi)	N	S	N	S	N	S	F	N	S	F
Min	42	38	41	38	42	38	38	38	38	42
Ave	51	52	50	52	51	52	52	51	52	49
Max	63	62	62	61	63	62	63	62	61	62

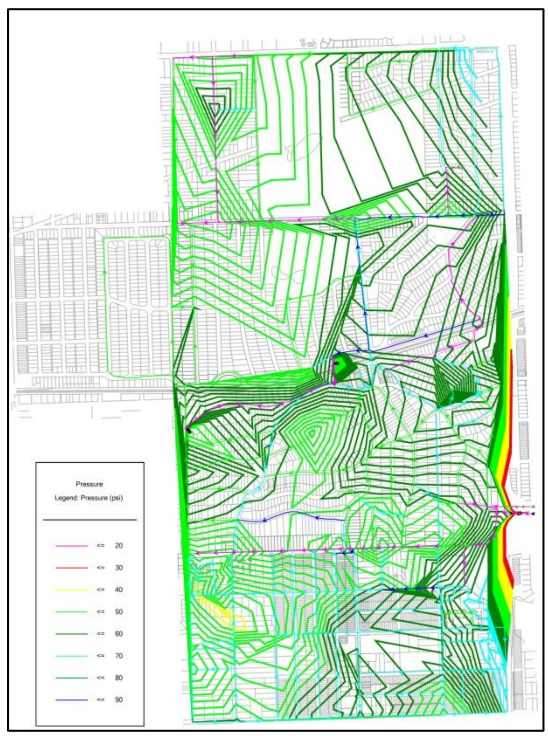
As **Table 4.5** shows, no pressures drop below 30 psi when the current pump model pressurizes the system. The assumed duty point for this analysis is 650 gpm at 55 psi. This is illustrated in **Figure 4.7** on the following page.

9. Current Condition – Average Day

10. Future Condition – Average Day



11. Current Condition – Average Day



12. Future Condition – Average Day

Figure 4.7: Pressure Results – Ann Street Pump Station Scenarios

4.5 SUMMARY OF WATER SYSTEM SHORTFALLS

4.5.1 Existing

Two system shortfalls were identified during the review of the existing conditions. The first shortfall pertains to storage volume and is discussed in **Section 4.5.1.1**. The second shortfall pertains to system pressures and is discussed in **Section 4.5.1.2**.

4.5.1.1 Emergency Storage Volume

Under Section 3 of the Village's agreement with the DuPage Water Commission, the following is defined with respect to storage:

> *Each Charter Customer agrees to use its best efforts to increase the water storage capacity of its Unit System; but in no event shall a Charter Customer be required to increase the water storage capacity contained in its Unit System (including its operable shallow well capacity which may be counted towards meeting up to 10% of its storage requirements) to an amount more than (i) twice the amount of its then annual average daily water demand, less that portion of the Commission's storage capacity in its Waterworks System . . .*

For the purposes of this Capacity Study, it has been assumed that the target storage volume for the Village is 2 times average daily demand, which also assumes that the feed from the Commission is cut-off and the Village's emergency wells are not utilized.

An assessment of the Village's Storage Volume is provided in **Table 4.6**.

Table 4.6: Emergency Storage Volume Calculation – Existing Condition

Volume / Demand	Calculation
Average Daily Demand	0.778 MGD
2 x Average Daily Demand	1.556 MG
Existing Storage Volume	1.250 MG
Emergency Volume Shortfall	**(0.306 MG)**

This calculation was also completed based on the Village's current operating settings, taking into account elevated reservoir set points, and pump limitations as discussed in **Section 2.3.2**.

Table 4.7: Emergency Operating Volume Calculation – Existing Condition

Volume / Demand	Calculation
Average Daily Demand	0.778 MGD
2 x Average Daily Demand	1.556 MG
Existing Operating Volume	0.580 MG
Emergency Volume Shortfall	**(0.976 MG)**

Although the emergency scenario is relatively conservative, **Tables 4.6 – 4.7** show that the existing emergency storage volume in the Village is low for the existing condition.

4.5.1.2 System Pressures

Two locations were identified as having pressure sensitivity in the Village. The first location is located on the North side of the BNSF railroad, bounded by Chicago Avenue on the north, Indian Drive on the East, Burlington Avenue on the south, and Hiawatha Drive on the east. During meetings with Village water operators, it was discussed that this area has old water mains in poor condition, and that pressures to the areas should remain below 45 psi. Although this area was not part of the scope of the Capacity Analysis, criteria was defined in the proposed alternatives to be mindful of this condition.

The second pressure sensitive location is the low pressure zone adjacent to Western Avenue, south of 55th Street. Due to elevations, system pressures in this area were shown to fall below 30 psi during average and peak demands. These two areas have been highlighted on **Figure 4.8** below for peak day scenario in the current condition.

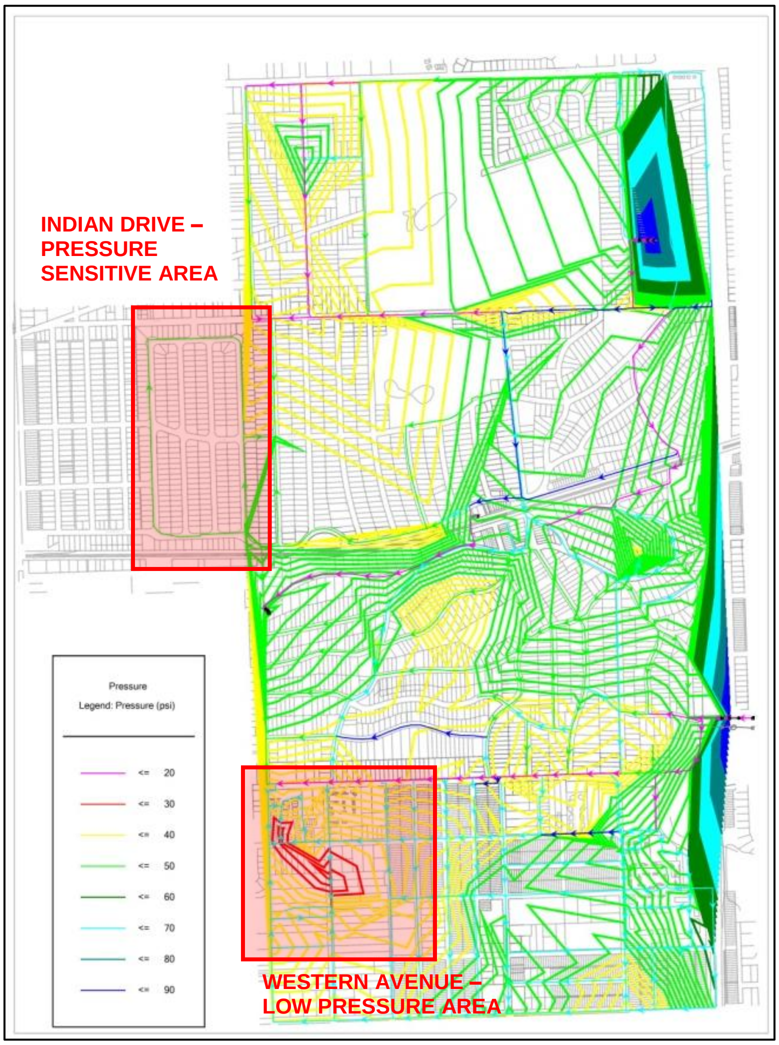


Figure 4.8: Pressure Sensitive Areas

These two areas have competing needs. In order to increase pressures in the Western Avenue area, system pressures need to increase. However, an increase in overall system pressures will lead to increased water main breaks in the Indian Drive area.

A further analysis was performed to see how the Western Avenue area correlates to elevated reservoir water levels. **Figure 4.9** illustrates that less than 10% of the elevated reservoir volume is capable of maintaining the minimum 30 psi pressure requirement.

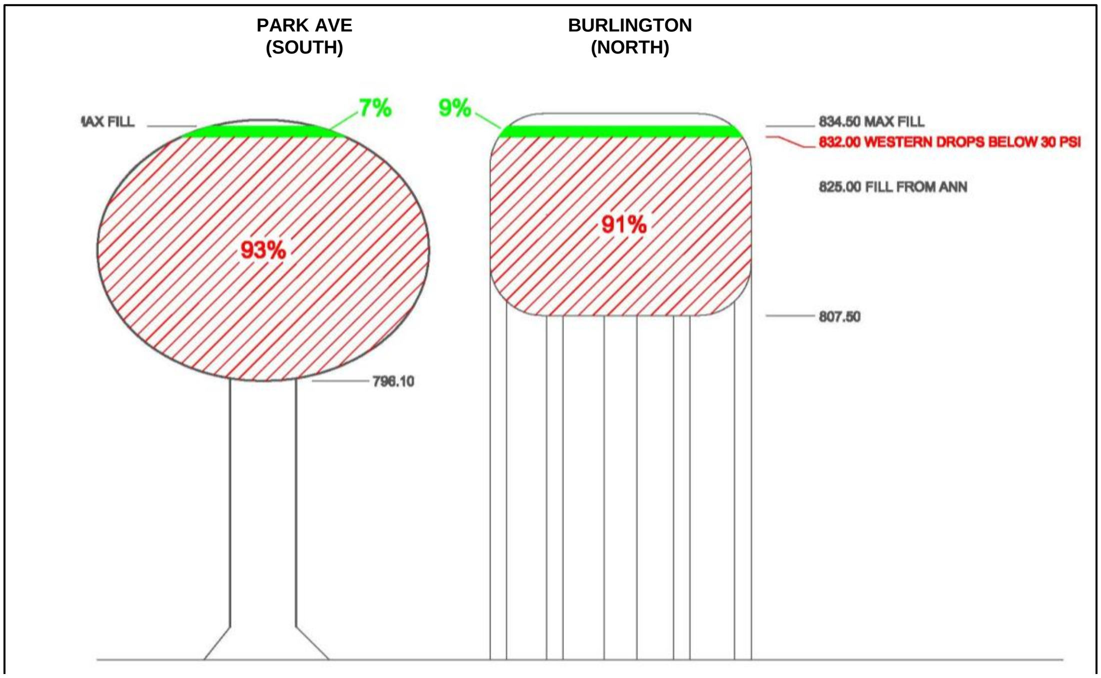


Figure 4.9: Elevated Reservoir Levels vs. Western Avenue Pressure

4.5.2 Future

The shortfalls identified in the existing condition are carried over into the future condition as discussed in **Sections 4.5.2.1 – 4.5.2.2**.

4.5.2.1 Emergency Storage Volume

Demands in the future condition are increased by approximately 10%, as defined in **Section 3.2.** The Emergency Storage Volume and Emergency Operating Volume calculations are provided in **Table 4.8** and **Table 4.9** respectively.

Table 4.8: Emergency Storage Volume Calculation – Future Condition

Volume / Demand	Calculation
Average Daily Demand	0.854 MGD
2 x Average Daily Demand	1.708 MG
Existing Storage Volume	1.250 MG
Emergency Volume Shortfall	**(0.458 MG)**

This calculation was also completed based on the Village's current operating settings, taking into account elevated reservoir set points, and pump limitations as discussed in **Section 2.3.2**.

Table 4.9: Emergency Operating Volume Calculation – Future Condition

Volume / Demand	Calculation
Average Daily Demand	0.854 MGD
2 x Average Daily Demand	1.708 MG
Existing Operating Volume	0.580 MG
Emergency Volume Shortfall	**(1.128 MG)**

4.5.2.2 System Pressures

System pressure shortfalls are exacerbated slightly more than those shown in **Figure 4.8** for the current condition due to the higher demand. **Figure 4.10** highlights the pressure problem areas for the future peak demand condition.

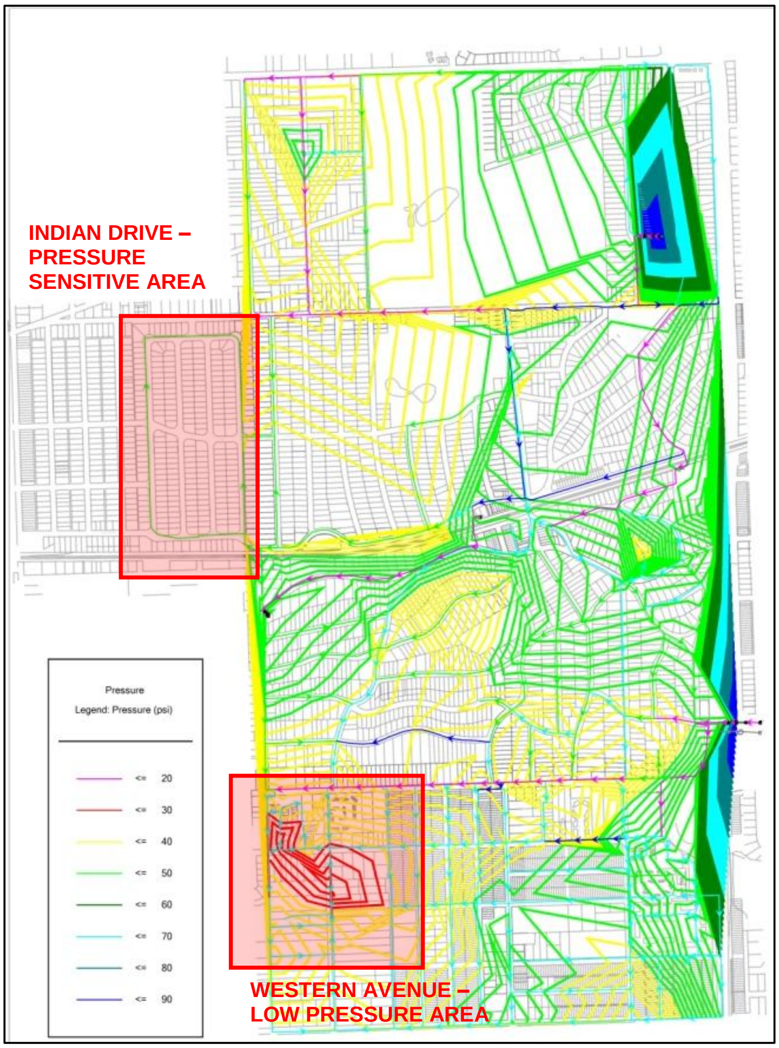


Figure 4.10: Pressure Sensitive Areas

CHAPTER 5 : ALTERNATIVE IMPROVEMENTS

Alternative Improvements were reviewed based on the development area and demands for the future condition. The goal of each alternative was to meet the following criteria:

1. Provide water storage equal to 2-days of average demand in the future condition.
2. Provide a minimum system pressure of 30 psi at all times during the 24 hour future peak demand simulation (when water is being delivered by the DuPage Water Commission).
3. Provide a minimum system pressure of 20 psi at all times during a 36 hour future average day emergency simulation (when water has been cut off by the DuPage Water Commission).
4. Limit pressures north of the BNSF Railroad to 50 psi.
5. Extend water main to all future development properties to the southern limits of the SSWSA.

These alternatives are described and analyzed in the subsequent sections.

5.1 ALTERNATIVE 1 – LOCAL PRESSURE ZONE CONTROL (ZONE BOOSTER)

5.1.1 Summary of Improvements

This alternative creates an isolated pressure zone for the area at and surrounding Western Avenue, south of 55th street. This pressure zone is approximately 0.09 square miles, bound by 55th Street on the north, Bentley Avenue on the east, 58th Street on the south, and Richmond Avenue extended on the west. The following improvements are required:

Table 5.1: Alternative 1 – Summary of Improvements

Type	Improvement	Description
Water Storage	0.5 MG Elevated Reservoir	A new elevated water storage tank will serve two purposes. First, the tank will add 0.5 MG to the Village's storage volume, bringing the total to 1.75 MG, meeting the 2 day average demand requirement. Second, the tank will maintain pressures in the low pressure zone to above 30 psi. The proposed location of the improvement is the 5700 block of Western Avenue.
Pumps	300 GPM Booster Pump	A booster station is required at the foot of the elevated reservoir to fill the tank to its maximum level since the pressure settings at the transfer stations are too low to achieve this without boosting. This will also be located on the 5700 block of Western Avenue.

Controls	4 Flow / Pressure Control Valves 2 Two-Way Altitude Valves	Combination Pressure / Flow Control Valves are required to isolate the pressure zone. Altitude valves are required at the two existing elevated reservoirs since the proposed reservoir will be higher than overflow elevations of these existing structures.
Distribution	21,000' – 8" D.I.W.M. (UTL Inc Areas) 14,000' – 8" D.I.W.M. (Private Areas) 2,100' – 12" D.I.W.M.	New 8-inch distribution main is required to extend the Village's distribution network into unincorporated areas. New 12-inch transmission main is required to feed the new booster pump station to fill the proposed 0.5 MG reservoir.

These improvements are illustrated in **Figure 5.1** below.

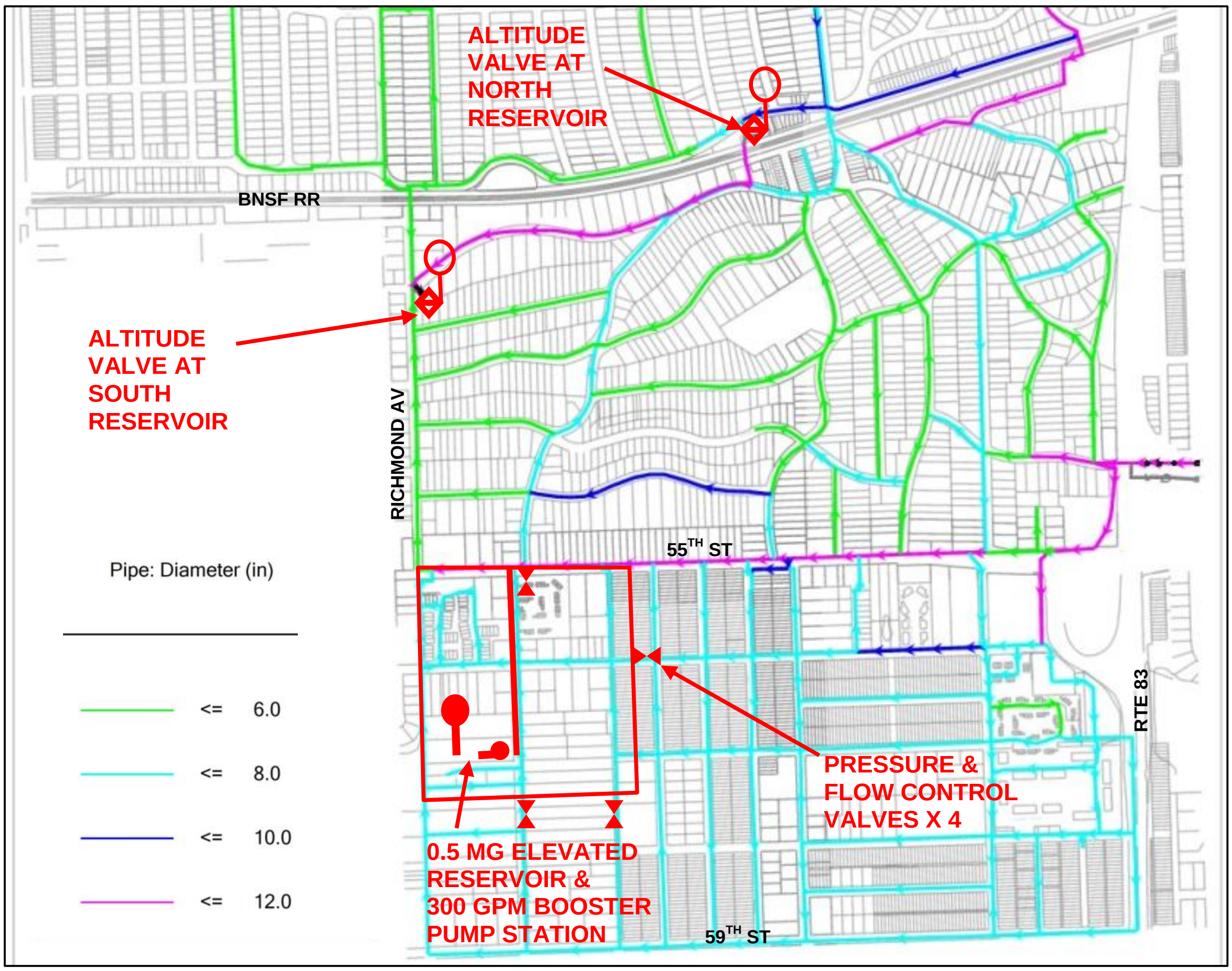


Figure 5.1: Alternative 1 Improvements

5.1.2 Assessment of Alternatives Criteria

Criteria 1 – Total Storage Volume
The addition of the 0.5 MG elevated reservoir increases the total storage volume to 1.75 MG in the Village compared to the 1.71 MG required.

Result: **Pass**

Criteria 2 – Minimum Pressure During Peak Demand
Pressure results at the end of the simulation (when reservoir levels are lowest) are summarized in **Table 5.2** below.

Table 5.2: Alternative 1 – Peak Demand System Pressures

Pressure (psi)	Alternative 1		
	North	South	Future
Min	28	29	29
Ave	37	34	37
Max	49	36	49

Only 5 out of 195 nodes had pressures below 30 psi, which can be addressed with the settings of the flow and pressure control valve fine tuning.

Result: **Pass**

Criteria 3 – Minimum Pressure During Emergency
Pressure results at the end of the simulation (when reservoir levels are lowest) are summarized in **Table 5.3** and **Figure 5.2** below.

Table 5.3: Alternative 1 – Emergency Service System Pressures

Pressure (psi)	Alternative 1		
	North	South	Future
Min	25	26	27
Ave	34	30	33
Max	46	33	46

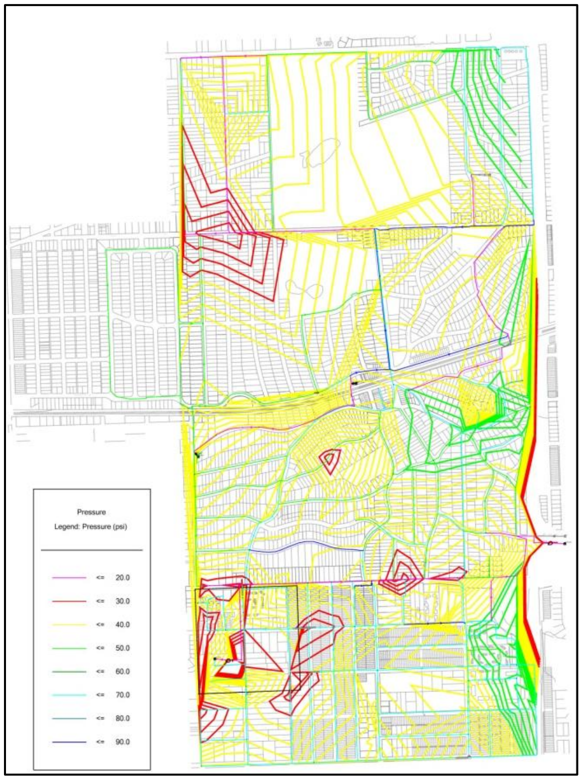


Figure 5.2: Alternative 1 – Emergency Service System Pressures

Result: **Pass**

Criteria 4 – North System Pressure Limitation
The range of pressures listed in **Tables 5.2 - 5.3** for the North System are 25 – 49 psi.

Result: **Pass**

Criteria 5 – Water Main Extension to Future Service Area
Approximately 7-miles of water main have been extended to meet this criteria.

Result: **Pass**

5.1.3 Discussion of Advantages and Disadvantages

The primary advantage of this Alternative is that the pressure settings at the transfer stations remain what they are currently. Pressures into the system therefore remain at levels comfortable to the Village operators, and should correlate with less water main breaks. Another advantage is that the Village will gain an additional 0.5 MG of elevated storage that is effective in delivering system pressures above 30 psi. As shown in Figures 4.5 – 4.6, system pressure can only rely on reservoir levels for a short period of time before dropping below this threshold.

Due to the local nature of the pressure zone, the SCADA and controls components of the system becomes more complicated. This is caused by the new elevated reservoir elevations being higher than the existing reservoirs. During the initial stages of implementation, reservoir overflows will be likely until the altitude and pressure / flow valves have been properly calibrated. This Alternative also introduces a second pump station, which adds to operation, maintenance, and system control.

5.1.4 Planning Estimate

Estimated costs for base construction and engineering have been prepared for planning purposes. The estimates source a variety of information and subsequently have a fairly wide accuracy range. A contingency of 20% has been added due to the very cursory nature of the summarized costs.

For Alternative 1, it has been assumed that the property for the proposed reservoir and booster pumps will be located on the 5700 block of Western Avenue. The estimate is summarized in **Table 5.4** below.

Table 5.4: Alternative 1 – Local Pressure Zone Control (Zone Booster) Planning Estimate

Component	Budget
0.5 MG Elevated Reservoir	$2,000,000
300 GPM Booster Pump Station	$750,000
Pressure & Flow Control Valve Installations	$200,000
Altitude Valve Installations	$100,000
SCADA System Improvements	$200,000
Distribution Water Main Improvements	$8,750,000
Transmission Water Main Improvements	$630,000
Construction Sub-Total	**$12,630,000**
Engineering / Administration 20%	$2,526,000
Contingency 20%	$2,526,000
Project Total	**$17,682,000**

5.2 ALTERNATIVE 2 – LOCAL PRESSURE ZONE CONTROL (ANN BOOSTER)

5.2.1 Summary of Improvements

Similar to Alternative 1, Alternative 2 creates an isolated pressure zone for the area at and surrounding Western Avenue, south of 55th street. Rather than a booster station inside the low pressure zone, Alternative 2 relies on the Ann Street Pump Station to provide higher pressures to fill the new elevated reservoir in the low pressure zone. The required improvements are listed in **Table 5.5**.

Table 5.5: Alternative 2 – Summary of Improvements

Type	Improvement	Description
Water Storage	0.5 MG Elevated Reservoir	A new elevated water storage tank will serve two purposes. First, the tank will add 0.5 MG to the Village's storage volume, bringing the total to 1.75 MG, meeting the 2 day average demand requirement. Second, the tank will maintain pressures in the low pressure zone to above 30 psi. The proposed location of the improvement is the 5700 block of Western Avenue.
Transfer Stations	Ann Booster Station Pump Improvements	The existing capacity of the Ann Street Pump Station is 650 GPM at 50 psi. In order to meet peak demands and fill reservoirs, this capacity will be increased to 1300 GPM at 60 psi.
Controls	4 Flow / Pressure Control Valves 2 Two-Way Altitude Valves 4 Pressure Reducing Valves	Combination Pressure / Flow Control Valves are required to isolate the pressure zone. Altitude valves are required at the two existing elevated reservoirs since the proposed reservoir will be higher than overflow elevations of these existing structures. Pressure reducing valves will be required to keep pressures north of the BNSF railroad below 50 psi during pumping periods.
Distribution	21,000' – 8" D.I.W.M. (UTL Inc Areas) 14,000' – 8" D.I.W.M. (Private Areas) 2,100' – 12" D.I.W.M.	New 8-inch distribution main is required to extend the Village's distribution network into unincorporated areas. New 12-inch transmission main is required to feed the new booster pump station to fill the proposed 0.5 MG reservoir.

These improvements are illustrated in **Figure 5.3** below.

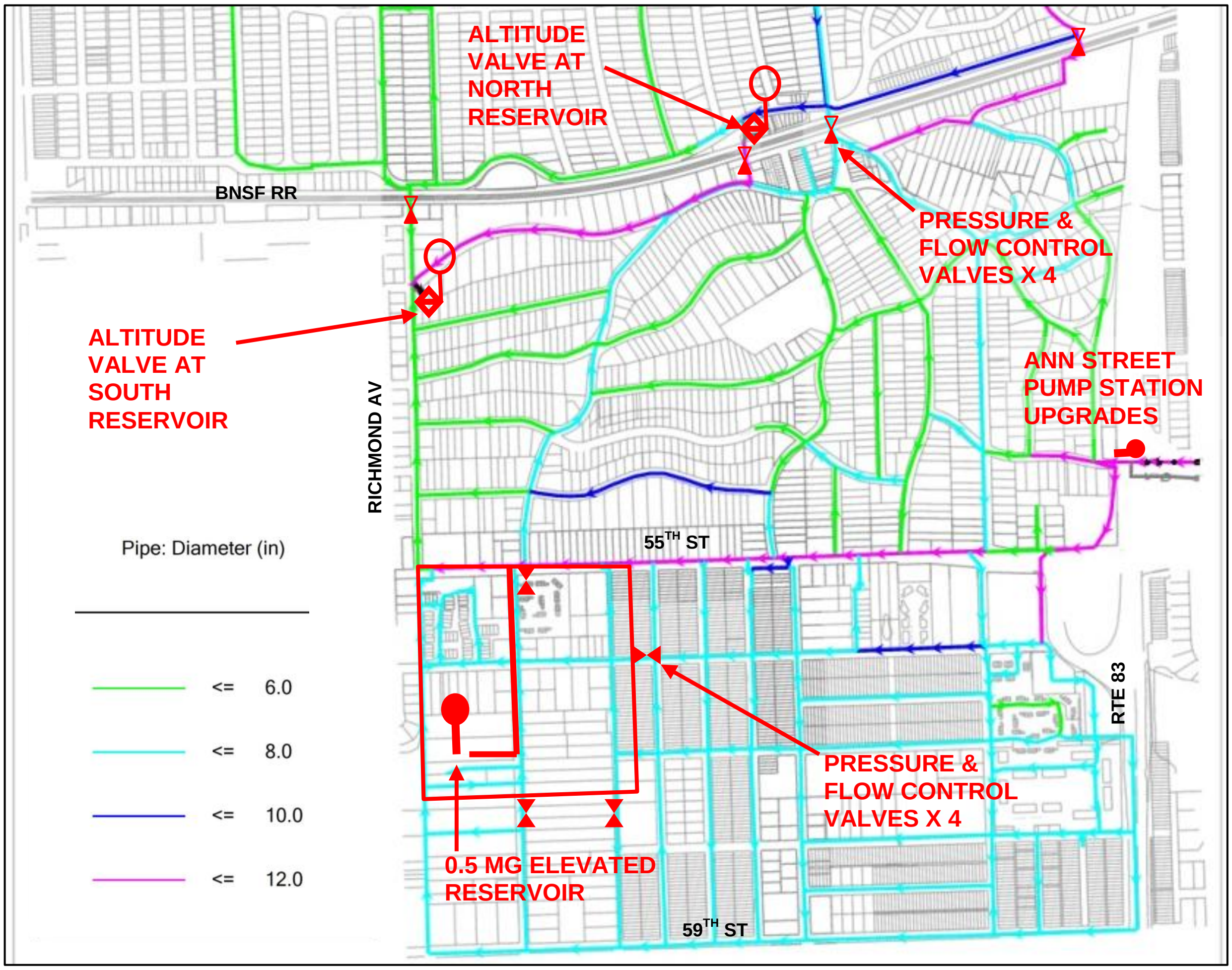


Figure 5.3: Alternative 2 Improvements

5.2.2 Assessment of Alternatives Criteria

Criteria 1 – Total Storage Volume

The addition of the 0.5 MG elevated reservoir increases the total storage volume to 1.75 MG in the Village compared to the 1.71 MG required.

Result: **Pass**

Criteria 2 – Minimum Pressure During Peak Demand

Pressure results are provided for two times during the simulation. The first time is 5 a.m., when the pump station is operating. The second time is 11:59 p.m. at the end of the simulation (when reservoir levels are lowest). Results are summarized in **Table 5.6** below.

Table 5.6: Alternative 2 – Peak Demand System Pressures

Pressure (psi)	Alt 2 (5 a.m.)			Alt 2 (11:59 p.m.)		
	N	S	F	N	S	F
Min	38	41	41	29	29	29
Ave	47	49	53	37	34	37
Max	59	54	67	44	36	49

All nodes remain above 30 psi. However, it should be noted that higher than normal system pressures are typical during the pump operations.

Result: **Pass**

Criteria 3 – Minimum Pressure During Emergency

Pressure results at the end of the simulation (when reservoir levels are lowest) are summarized in **Table 5.7** and **Figure 5.4** below.

Table 5.7: Alternative 2 – Emergency Service System Pressures

Pressure (psi)	Alt 2		
	N	S	F
Min	24	0	0
Ave	33	22	29
Max	40	31	45

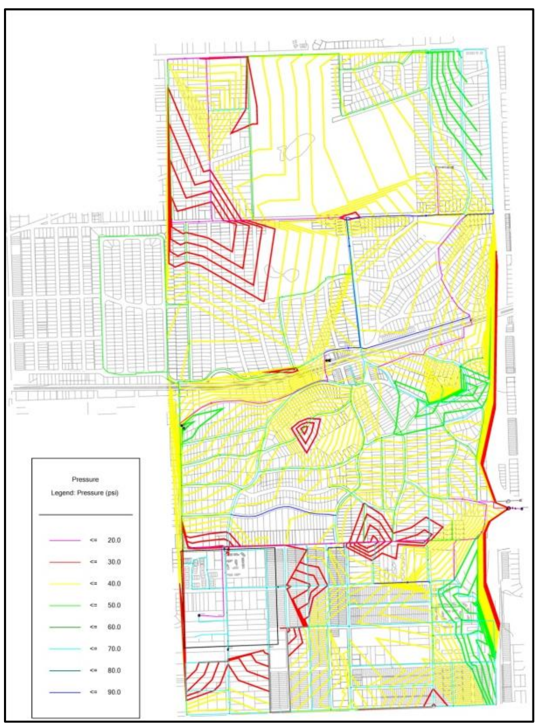


Figure 5.4: Alternative 2 – Emergency Service System Pressures

The data illustrates that once the new elevated reservoir empties, the system depends on the water levels in the existing elevated reservoirs. By the end of the 36-hour simulation, these levels are not capable of supplying pressure to the low pressure zone on Western Avenue.

Result: **Fail**

Criteria 4 – North System Pressure Limitation
The range of pressures listed in Tables 5.6 – 5.7 for the North System are 24 – 59 psi. Alternative 2 is prone to higher pressures due to the pressure increase in the pump station at Ann Street.

Result: **Fail**

Criteria 5 – Water Main Extension to Future Service Area
Approximately 7-miles of water main have been extended to meet this criteria.

Result: **Pass**

5.2.3 Discussion of Advantages and Disadvantages

The primary advantage of this Alternative is that no new pump station facilities are required, although upgrades are required at the Ann Street Pump Station. Furthermore, the pressure settings at the transfer stations remain where they are at currently. Another advantage is that the Village will gain an additional 0.5 MG of elevated storage, used to meet the 2-day average flow storage requirement for the future condition.

The main disadvantage is the increased pressures into the system. This may lead to increased water main breaks and requires additional pressure reducing valves in order to minimize the north system pressures, effectively creating two new pressure zones rather than the single low pressure zone as in Alternative 1. This Alternative requires the pressure and capacity to be increased at Ann Street in order to fill the new elevated reservoir. This is a less economical approach that will lead to higher operating costs.

Due to the local nature of the pressure zone, the SCADA and controls components of the system become more complicated. This is caused by the new elevated reservoir elevations being higher than the existing reservoirs. During the initial stages of implementation, reservoir overflows will be likely until the altitude and pressure / flow valves have been properly calibrated.

Finally, the modelling showed that the low pressure zone at Western Avenue will lose pressure before 36 hours during an emergency condition.

5.2.4 Planning Estimate

It has been assumed that the property for the proposed reservoir is on the 5700 block of Western Avenue and that improvements at all other operating facilities are confined to Village property.

Table 5.8: Alternative 2 – Local Pressure Zone Control (Ann Booster) Planning Estimate

Component	Budget
0.5 MG Elevated Reservoir	$2,000,000
Ann Street Booster Pump Station Upgrades	$500,000
Pressure & Flow Control Valve Installations	$200,000
Altitude Valve Installations	$100,000
Pressure Reducing Valve Installations	$200,000
SCADA System Improvements	$200,000
Distribution Water Main Improvements	$8,750,000
Transmission Water Main Improvements	$630,000
Construction Sub-Total	**$12,580,000**
Engineering / Administration 20%	$2,516,000
Contingency 20%	$2,516,000
Project Total	**$17,612,000**

5.3 ALTERNATIVE 3 – WESTERN AVENUE SYSTEM AND ANN STREET STORAGE

5.3.1 Summary of Improvements

Unlike Alternatives 1 and 2, Alternative 3 proposes to create a separate water network specifically for the Western Avenue low pressure area, with a separate storage supplement at the Ann Street station for the greater network. Alternatives 1 and 2 proposed an elevated reservoir that had the dual purpose of serving the Western Avenue low pressure zone, while also offering additional storage for the entire system with a wider elevation operating range. Alternative 3 allows the existing system to operate in a similar fashion as it does currently, but also adds a separate smaller system in the Western Avenue Low Pressure Zone with its own set of operating conditions. The required improvements are listed in **Table 5.9**.

Table 5.9: Alternative 3 – Summary of Improvements

Type	Improvement	Description
Water Storage	70,000 Gallon Ground Reservoir (5700 Block, Western Avenue)	A new ground storage tank will serve the Western Avenue Low Pressure Area, sized to provide 2 day average demand to this specific location. The proposed location of the improvement is the 5700 block of Western Avenue.
Water Storage	0.4 MG Gallon Ground Reservoir (Ann Street Transfer Station)	A new ground storage tank will add 0.4 MG to the Village's storage volume, bringing the total to 1.75 MG, meeting the 2 day average demand requirement. The proposed location of the improvement is the Ann Street Transfer Station.

Pump Station	Western Avenue Pump Station	New pump station will serve the Western Avenue Low Pressure Area. Capacity is approximated at 50 GPM at 50 psi.
Controls	4 Flow / Pressure Control Valves	Combination Pressure / Flow Control Valves are required to connect the low pressure zone to the larger distribution network in case of a fire flow condition, and as a redundant source should the Western Avenue Pump Station be offline.
Distribution	21,000' – 8" D.I.W.M. (UTL Inc Areas) 14,000' – 8" D.I.W.M. (Private Areas) 2,100' – 12" D.I.W.M.	New 8-inch distribution main is required to extend the Village's distribution network into unincorporated areas. New 12-inch transmission main is required to fill the new Western Avenue 70,000 Gallon Ground Reservoir.

These improvements are illustrated in **Figure 5.5** below.

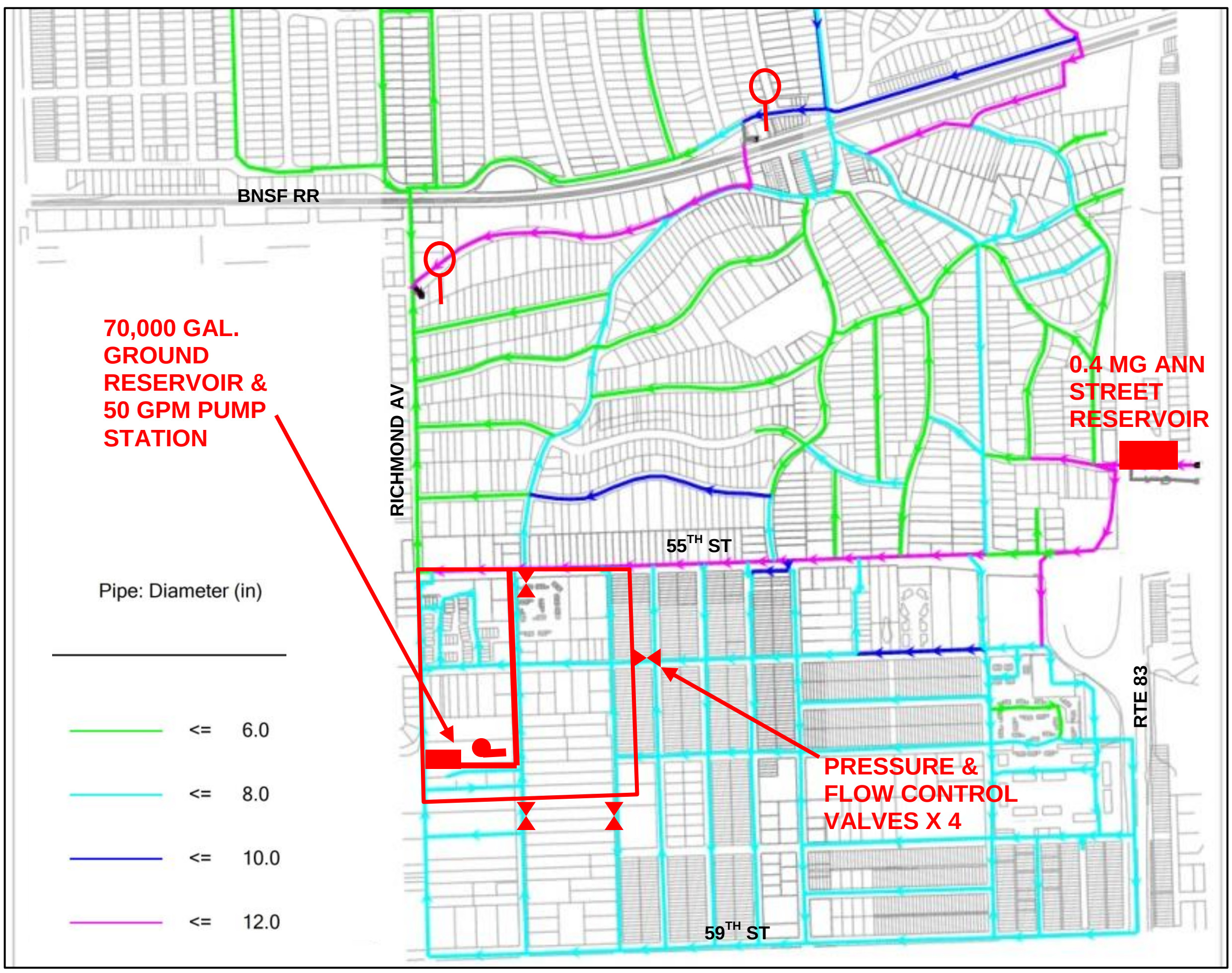


Figure 5.5: Alternative 3 Improvements

5.3.2 Assessment of Alternatives Criteria

<u>Criteria 1 – Total Storage Volume</u>
The addition of the 700,000 gallon Western Avenue ground reservoir and 0.4 MG Ann street ground reservoir increases the total storage volume to 1.75 MG in the Village compared to the 1.71 MG required.

Result: **Pass**

<u>Criteria 2 – Minimum Pressure During Peak Demand</u>
Pressure results at 7:30 a.m. (when reservoir levels are lowest) are summarized in **Table 5.10** below.

Table 5.10: Alternative 3 – Peak Demand System Pressures

Pressure (psi)	Alt 3		
	N	S	F
Min	28	28	28
Ave	37	33	36
Max	49	36	48

Only 7 out of 195 nodes had pressures below 30 psi, which can be addressed with the settings of the flow and pressure control valve fine tuning, and reservoir set-points.

Result: **Pass**

<u>Criteria 3 – Minimum Pressure During Emergency</u>
Pressure results at the end of the simulation (when reservoir levels are lowest) are summarized in **Table 5.11** and **Figure 5.5** below.

Table 5.11: Alternative 3 – Emergency Service System Pressures

Pressure (psi)	*Alt 3*		
	N	*S*	*F*
Min	*19*	*20*	*20*
Ave	*28*	*25*	*31*
Max	*41*	*28*	*64*

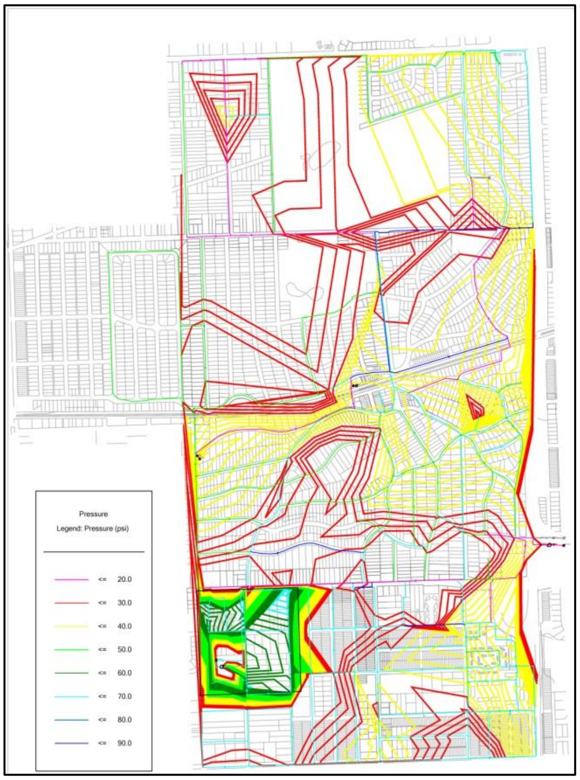


Figure 5.5: Alternative 3 – Emergency Service System Pressures

Result: **Fail**

Criteria 4 – North System Pressure Limitation
The range of pressures listed in **Tables 5.10 - 5.11** for the North System are 19 – 49 psi.

Result: **Pass**

Criteria 5 – Water Main Extension to Future Service Area
Approximately 7-miles of water main have been extended to meet this criteria.

Result: **Pass**

5.3.3 Discussion of Advantages and Disadvantages

The primary advantage of this Alternative is that the majority of the system will operate based on the current Village methodology, floating on the existing elevated reservoirs. The pressure settings at the transfer stations remain where they are at currently. Another advantage is that the Village will gain an additional 0.5 MG of ground storage, used to meet the 2-day average flow storage requirement for the future condition. This alternative also offers construction phasing opportunities. The Western Avenue pump station and reservoir can be constructed earlier in the planning phase to address the immediate low pressure concern. This gives the

Village flexibility to gauge the pace of development, and the need for the ground storage reservoir at the Ann Transfer Station.

The main disadvantage is that many of the pressure issues identified in Section 4.5 remain unresolved. During an emergency Scenario, without a feed from the DuPage Water Commission, pressures in the entire system are impacted by the low elevated reservoir elevations. Although some operational refinements can be made so that the system is more reliant on Ann Street pump pressures, it is a fine line, and low pressures will be likely during and emergency situation.

A new independent system is created for the low pressure zone at Western Avenue, which requires its own independent operational settings. Thus the Village is almost divided into two independent systems. The new Western Avenue system also presents fire flow challenges. The pump station would be sized for peak future demands only, and would thus be reliant on the outside network to supplement for fire flows.

5.3.4 Planning Estimate

It has been assumed that the property for the proposed improvements are either the property of the Village or in the right of way.

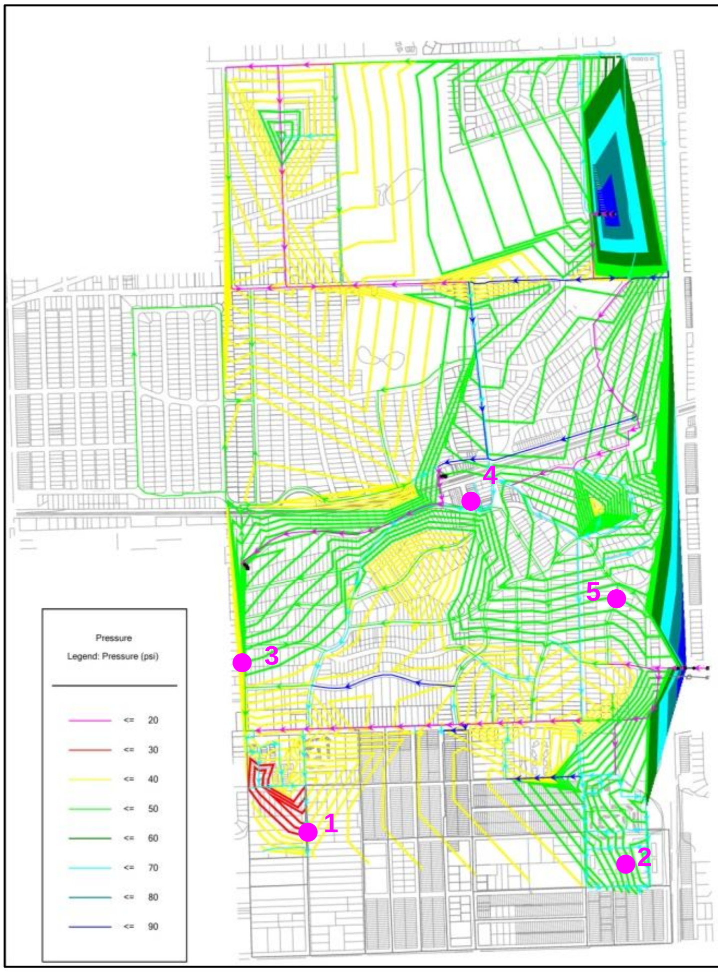
Table 5.8: Alternative 3 – Western Avenue System & Ann St. Storage Planning Estimate

Component	Budget
70,000 Gallon Ground Storage Tank	$200,000
Western Avenue Pump Station	$400,000
Pressure & Flow Control Valve Installations	$200,000
0.4 MG Ground Storage Tank	$2,000,000
SCADA System Improvements	$200,000
Distribution Water Main Improvements	$8,750,000
Transmission Water Main Improvements	$630,000
Construction Sub-Total	**$12,380,000**
Engineering / Administration 20%	$2,476,000
Contingency 20%	$2,476,000
Planning Estimate	**$17,332,000**

CHAPTER 6 : FIRE FLOW ANALYSIS

Fire flow analysis was performed for the existing and future conditions based on the current water system infrastructure, in order to highlight system vulnerabilities. The approach for this study was to simulate a 3-hour fire in 5 locations. As the development planning process progresses, it is recommended that more detailed fire flow analysis is carried out. The approach for this report is intended to provide a general feel for fire flows in the system.

At hour 3, the system pressures in the SSWSA was reviewed for minimum and average pressures compared to the required minimum pressure of 20 psi. The five locations are summarized in **Figure 6.1** below, shown on the existing pressure contour map for the existing peak demand scenario.



<u>Test 1</u>
Location: Western Avenue
Junction: J-55
Property: Single Family Residential
Fire Flow: 1,000 gpm – 3 hr

<u>Test 2</u>
Location: Carlysle Drive
Junction: J-134
Property: Multi Family Residential
Fire Flow: 1,500 gpm – 3 hr

<u>Test 3</u>
Location: Richmond Ave. & Hudson Ave.
Junction: J-88
Property: Single Family Residential
Fire Flow: 1,000 gpm – 3 hr

<u>Test 4</u>
Location: Walker Avenue
Junction: J-186
Property: Downtown Business District
Fire Flow: 1,500 gpm – 3 hr

<u>Test 5</u>
Location: Ann St. & Powell St.
Junction: J-30
Property: Single Family Residential
Fire Flow: 1,000 gpm – 3 hr

Figure 6.1: Fire Flow Test Locations

Fire flow results for the five test locations are shown in **Figure 6.2** for Existing Conditions and **Figure 6.3** for Future Conditions**.**

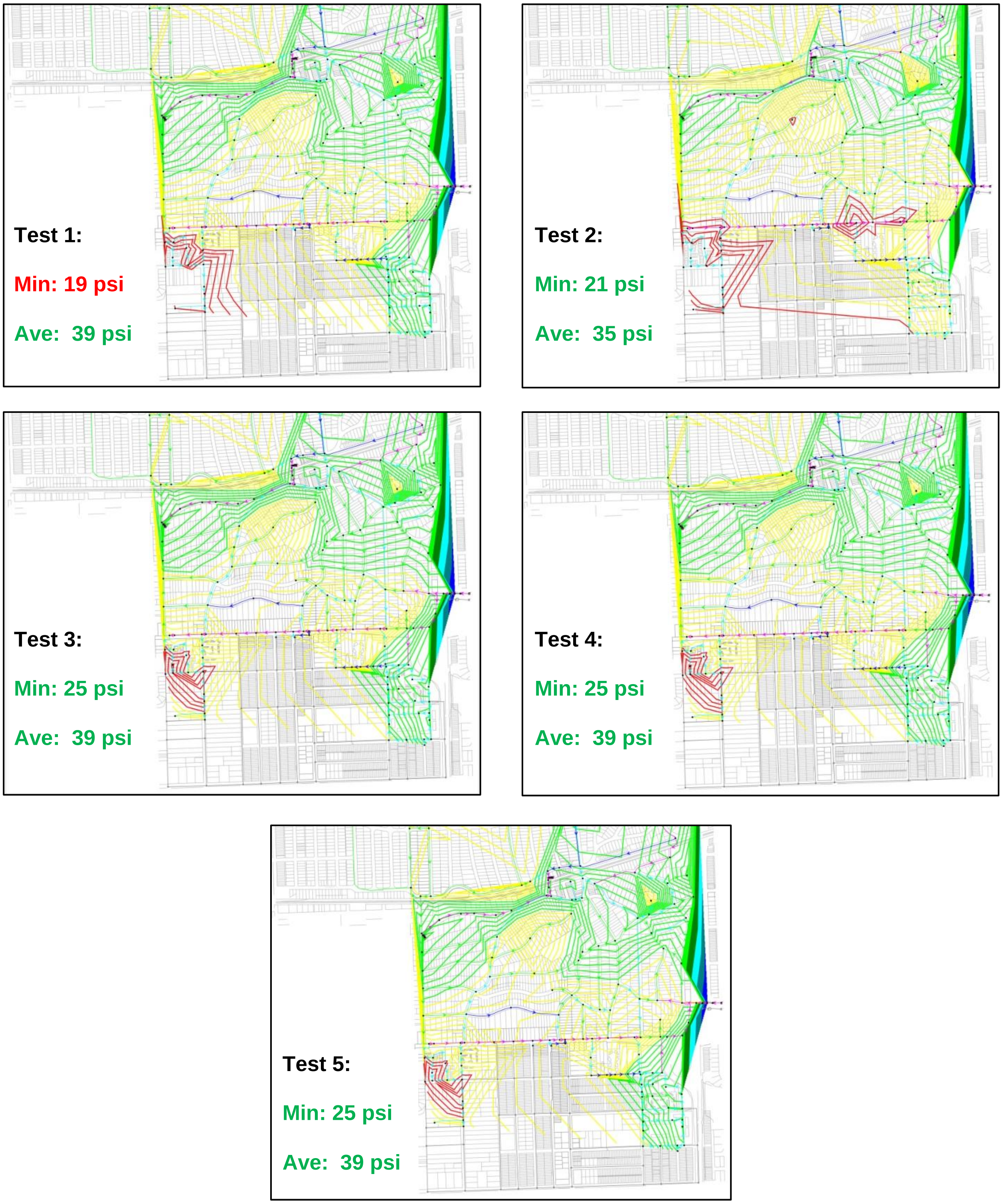


Figure 6.2: Fire Flow Test Results – Existing Conditions

Test 1:

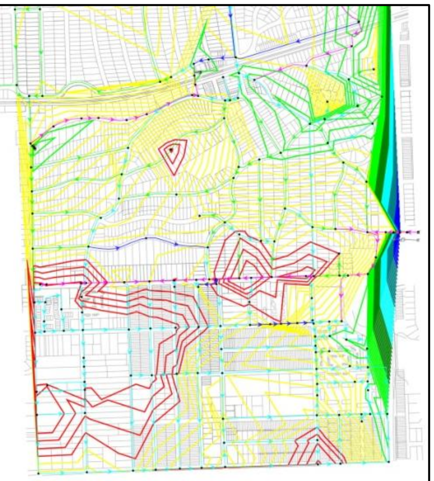
Min: 22 psi

Ave: 37 psi

Test 2:

Min: 19 psi

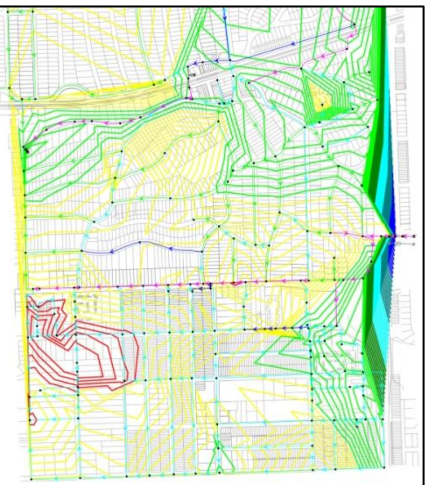
Ave: 33 psi



Test 3:

Min: 24 psi

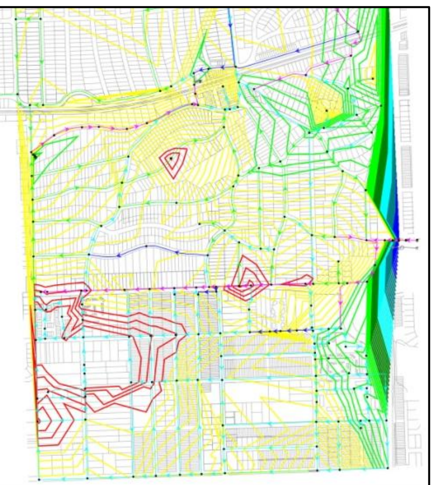
Ave: 38 psi



Test 4:

Min: 21 psi

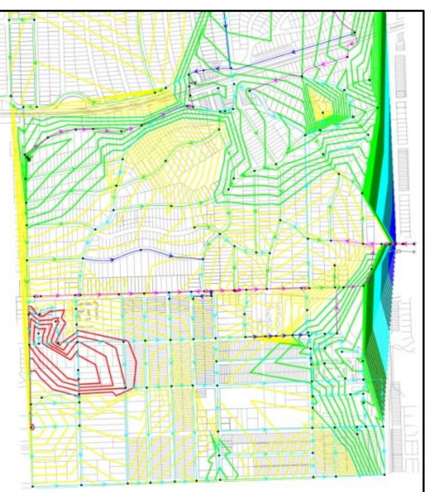
Ave: 35 psi



Test 5:

Min: 24 psi

Ave: 38 psi



The fire flow analysis highlights the vulnerability of the Western Avenue Low Pressure Area. The minimum system pressures were identified on Western Avenue in all instances, including the two measurements of 19 psi, which are below the required minimum of 20 psi. As development planning progresses, it is recommended that a more thorough fire flow analysis be carried out in order to ensure appropriate fire flows and system pressures are provided for specific development scenarios.

Figure 6.3: Fire Flow Test Results – Future Conditions

CHAPTER 7 : GOVERNMENTAL CONSIDERATIONS

Some additional considerations were reviewed in general, with regards to future water system improvements. These include a review of the general characteristics of the neighboring municipalities of Willowbrook and Westmont, as well as the viability of the take-over of the UTL Clarendon Water Company Infrastructure.

7.1 WILLOWBROOK AND WESTMONT

The Village of Clarendon Hills currently shares emergency interconnections with the Villages of Willowbrook and Westmont at the following locations.

Table 7.1: Emergency Interconnect Locations

Municipality	Location	Size	Ground Elevation
Village of Willowbrook	Holmes Avenue and 58th Street	8”	± 738
Village of Westmont	Richmond Avenue and Quincy Street	6”	± 738

Available public water system information was reviewed for both municipalities in order to gauge how the water systems might interact. This general review included a comparison of elevations, demands, and water storage.

7.1.1 Elevations

Figure 7.1 provides a general view of the elevations across the 3 communities. Municipal boundary lines have been simplified.

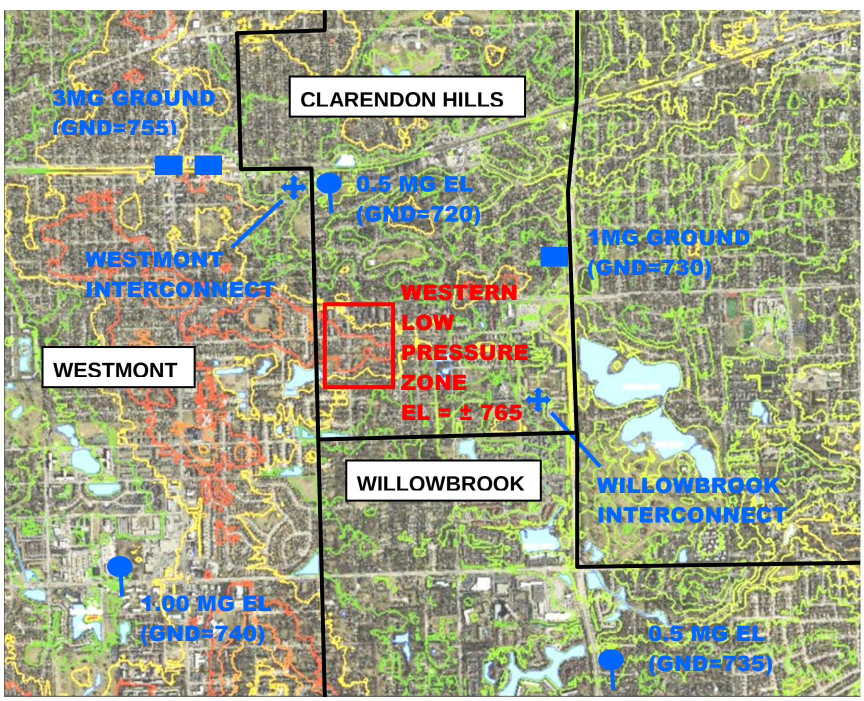


Figure 7.1: Adjacent Municipalities

Based on a review of the elevations, it appears that the Village of Westmont is more favorable with respect to the Village of Clarendon Hills current and future pressure issues along Western Avenue. The Westmont ground storage tanks lie along the same high topography band as Western Avenue, and would likely involve less substantial improvements to achieve appropriate pressures at the Western Avenue Low Pressure Zone, compared to Willowbrook.

7.1.2 Demands

In order to gauge the water demand for the adjacent communities, the latest public IDNR LMO-2 reporting was reviewed for the Villages of Willowbrook and Westmont. These allocations and demands are summarized in **Table 7.2** below.

Table 7.2: 2011 Westmont and Willowbrook LMO-2 Demand Reporting

Municipality	Allocation (MGD)	Net Annual Pumpage (MGD)	Surplus (MGD)	Surplus (%)
Village of Willowbrook	1.286	1.005	0.281	22%
Village of Westmont	2.957	2.353	0.604	20%

Based on the 2011 reporting, both Villages appeared to have a surplus in their allocation. Additional considerations with the communities would include increased growth since 2011, future development considerations, and annexation plans, among others.

7.1.3 Water Storage

Excess water storage is another factor that was reviewed to gain an understanding of the water system capacities of the neighboring communities. The water storage volumes of the Village of Willowbrook and Westmont are provided in **Tables 7.3 and 7.4** respectively.

Table 7.3: Willowbrook Water Storage Facilities

Location	Type	Volume (MG)
67th St & Route 83	Elevated	0.5
Quincy St & 75th St	Standpipe	3.0
Quincy St & Midway Dr	Elevated	0.5
Total		**4.0**
Ave Daily Demand		1.0
Emergency Surplus		**3.0**

Table 7.4: Westmont Water Storage Facilities

Location	Type	Volume (MG)
Burlington Ave & Linden Ave	Ground	3.0
63rd St & Suffield Ct	Elevated	1.0
Total		**4.0**
Ave Daily Demand		2.4
Emergency Surplus		**1.6**

Based on a review of the water storage facility tables, it appears at first glance that the Village of Willowbrook has a more robust emergency water storage volume reserve.

7.1.4 Summary

It must be pointed out that the Villages of Willowbrook and Westmont were not approached while compiling this report. The information presented in **Section 7.1** is based upon public information only and does not include factors such as emergency well supply, water system operations, and other considerations. The information presented should be used only as an initial gauge of the water system capacities of the neighboring communities, for the purposes of further investigation involving participation by Willowbrook and Westmont staff.

The Village of Willowbrook appears to have more storage reserves in their system, which may be a consideration when looking at a shared supply situation. Elevations however, would likely dictate the need for additional pump station facilities in order to achieve the goals of the SSWSA.

The Village of Westmont has operating facilities in closer proximity to the SSWSA, and has a primary operating facility at favorable elevations. The pressures between the two systems would likely be easier to coordinate. That being said, a partnership with the Village of Westmont would likely require an additional shared storage facility to address the needs of both Villages.

7.2 UTL INC – CLARENDON WATER COMPANY

The UTL Inc – Clarendon Water Company currently supplies water to high density residential properties in the south-central area of the SSWSA. In contrast to the Village's Lake Michigan water, the private utility sources water from shallow wells, via distribution pressure storage / filtration system. The utility became active in 1965 and serves approximately 900 people according to available public information. No atlas information was available, but it has been assumed that the water mains of this private system are sized between 4" – 6". The fire hydrants visible at street level do not conform to the standards required by the Village.

Should community development expand into the territory of the private utility, it is important the Village has an understanding of the system in case inquiry is made by the ICC or IEPA.

The approximate limits of the utility have been highlighted in **Figure 7.2** below. It is estimated that utility owns and operates approximately 4 miles of watermain.

Figure 7.2: UTL Inc – Clarendon Water Company Infrastructure

Due to the unknowns of the system, and the assumed sub-standard diameters and hydrants, it is not recommended that the Village pursue a take-over of the infrastructure of this private utility. A quick analysis shows a minor drop in the average system pressure south of 55th Street, however, the smaller mains will have lower fire flows and the condition of the infrastructure is unknown.

CHAPTER 8 : DECISION MATRIX OF IMPROVEMENTS

A decision matrix has been created that assigns a score to each alternative based upon a list of criteria. The purpose of this matrix is to quantify each alternative such that informed decisions can be made during planning and development. Major factors have been identified that will impact the success of the future water capacity infrastructure. These factors are labelled across the top of the matrix and given a weighting percentage in accordance with defined criteria. For example capital cost has been given a weight of 30% while operation and maintenance has been given a weight of 10%. The alternatives are listed on the left side. For each alternative, a number from 1 to 10 has been assigned under each factor based on its relationship to that factor and multiplied with the weighted percentage. For example, if a given option has a high capital cost, it will be assigned a low number times the weighting. This will be done for each factor until a total is determined for each route. The Alternative with the highest number will (numerically at least) identify the best alternative, considering all of the factors necessary for an informed decision.

Table 8.1: Decision Matrix of Alternatives

Alternative (Weight %)	Perform. & Operations (30%)	Capital Cost (30%)	Phasing (20%)	O&M (10%)	Gov / Leg (10%)	Score	Rank
Alternative 1	8	5	5	4	5	**5.8**	**1**
Alternative 2	4	5	5	7	5	**4.9**	**3**
Alternative 3	5	6	7	4	5	**5.6**	**2**

Based on the matrix value assignments, Alternative 1 ranks the highest. This tool should be reviewed and modified by the Village as needed based upon changing priorities as development planning process progresses.