City of Portage

Water System Reliability Study

Prepared for:

City of Portage, Michigan

October 21, 2013
Project No. G120751

fTC&h

CITY OF PORTAGE

WATER SYSTEM RELIABILITY STUDY

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LIST OF ABBREVIATIONS/ACRONYMS

City	City of Portage, Michigan
FTC&H	Fishbeck, Thompson, Carr & Huber, Inc.
GIS	Geographic Information System
gpcd	gallons per capita per day
gpd	gallons per day
gpm	gallons per minute
lft	linear foot
MCL	maximum contaminant level
MDEQ	Michigan Department of Environmental Quality
MG	million gallons
mgd	million gallons per day
psi	pounds per square inch
TDH	total dynamic head
USEPA	United States Environmental Protection Agency
VFD	variable frequency drive
σ	standard deviation

1.0 EXECUTIVE SUMMARY

The City of Portage (City) operates a municipal water distribution system that supplies water to its residents and businesses. This report assesses the reliability of the water system and its ability to supply demands, and will assist the City in planning for future growth.

Water is pumped from City-owned wells to supply system demands. System demands are projected to increase as the City population increases. Average day demands are projected to increase from 5.86 million gallons per day (mgd) in 2013 to 6.98 mgd in 2033. Maximum day demands are projected to increase from 17.05 mgd in 2013 to 20.31 mgd in 2033.

The firm capacity of the water supply is currently 23.79 mgd, which can adequately supply projected 2013 and 2033 demands. The firm capacity is likely to be reduced to 20.92 mgd when Winterforest and Portage Creek Wells are taken out of service in the future. An arsenic, iron, and manganese removal plant was built to treat the water from Garden Lane Wells 1 through 4, allowing these wells to continue to be used despite elevated arsenic levels at Garden Lane 1 and 2. Additional water supply is projected to be necessary near the end of the 20-year planning period to adequately supply maximum day demands.

A hydraulic model of the water system was updated based on data in the City's Geographic Information System (GIS) system using computer modeling software. The model was calibrated based on field data and was used to evaluate the condition of the water system.

The City water distribution system is in good condition, and benefits from a large amount of new, large-diameter piping. Available fire flows and pressures across the system are generally very good. There are some isolated areas where improvements are needed to increase available fire flows to greater than the minimum desired 1,000 gallons per minute (gpm). System pressures are generally adequate across the system. There is a small area in the vicinity of Westfield Park where pressures can fall to less than 35 pounds per square inch (psi) due to the high relative elevation of this area. Proposed pipe improvements were evaluated using the hydraulic model.

Elevated storage capacity was also evaluated. The analysis determined that additional elevated storage is needed in the near future. One new 2.5 million gallon (MG) tank in the Haverhill Park area to replace the existing Haverhill Tank is recommended. Additional storage should also be considered to meet the criteria of 25% of maximum day demand. The Pineview Tank interior should be re-coated.

The total cost to build one 2.5 MG tank is estimated at $5,928,000. The cost to coat the interior of the Pineview Tank is estimated at $350,000. The total cost of the recommended distribution system improvements to improve fire flows to 1,000 gpm or greater across the entire system, is estimated at $603,000. If a higher level of fire protection is desired, an expanded set of improvements to increase fire flows to 1,500 gpm or greater could be implemented at an estimated cost of $874,000. These costs are given in year 2013 dollars.

2.0 INTRODUCTION

The Part 12 Rules of the Michigan Safe Drinking Water Act (Act 399 P.A. of 1976 as amended) require that communities having Type I water supplies conduct a water system reliability study. State of Michigan administrative rules require the completion of a reliability study every five years, focusing primarily on the ability of the system to meet present and projected future water demands.

In addition to the basic state requirements for a reliability study, a calibrated hydraulic model of the water distribution system was in accordance with the Part 16 Rules of the Michigan Safe Drinking Water Act (Act 399 P.A. of 1976 as amended) for a general plan. The hydraulic model of the water distribution system developed as part of the previous reliability study was updated, calibrated, and used to identify deficiencies and evaluate improvements required to provide reliable water service into the future. The Water System Reliability Study provides a comprehensive review of the water system. This report is intended to fulfill the requirements of the State of Michigan for a reliability study and general plan.

3.0 HISTORICAL WATER USE

3.1 AVERAGE ANNUAL AND PER CAPITA PUMPAGE

Historical water pumpage data was compiled and reviewed for the past 10 years, 2003 through 2012. The total annual usage, average day usage, and per capita usage for this period are shown in Table 1. Population figures were estimated using year 2000 and 2010 U.S. Census Bureau Data. Two known values of the percentages of population served were used to estimate the percentage of population served for the past 10 years. The first known point was in 1990; the percentage of the population served was 52% as reported in the *Water Utility Infrastructure Analysis Program Report, City of Portage* (Snell Environmental Group [SEG] April 1995). The second known point was in 2000; the percentage of population served was approximately 90% as reported by the City. Data for the years between these points was interpolated. It was assumed that the connected percentage has not and will not increase above 90%. The values in this report were discussed with the City and are believed to be reasonable estimates of the percentage of the population served.

Table 1 – Summary of Pumpage for 2003 Through 2012

Year	Average Day Pumpage (mgd)	Total Population	Estimated Percentage of Population Served	Estimated Serviced Population	Approximate Average Per Capita Pumpage (gpcd)
2003	5.495	45,125	82.9%	37,648	146.0
2004	5.227	45,201	85.3%	38,878	134.5
2005	5.950	45,277	87.6%	39,674	150.0
2006	5.273	45,480	90.0%	40,932	128.8
2007	6.359	45,683	90.0%	41,115	154.7
2008	5.812	45,886	90.0%	41,297	140.7
2009	5.509	46,089	90.0%	41,480	132.8
2010	4.903	46,292	90.0%	41,663	117.7
2011	4.967	46,836	90.0%	42,152	117.8
2012	5.764	47,380	90.0%	42,642	135.2
Average	5.526	--	--	--	135.8
Maximum	6.359	--	--	--	154.7
Minimum	4.903	--	--	--	117.7

gpcd = gallons per capita per day

Figure 1 shows a graphical representation of annual average day pumpage over the period. Average day pumpage for the past 10 years has slowly decreased. However, the trend over the past 20 years has been a steady increase.

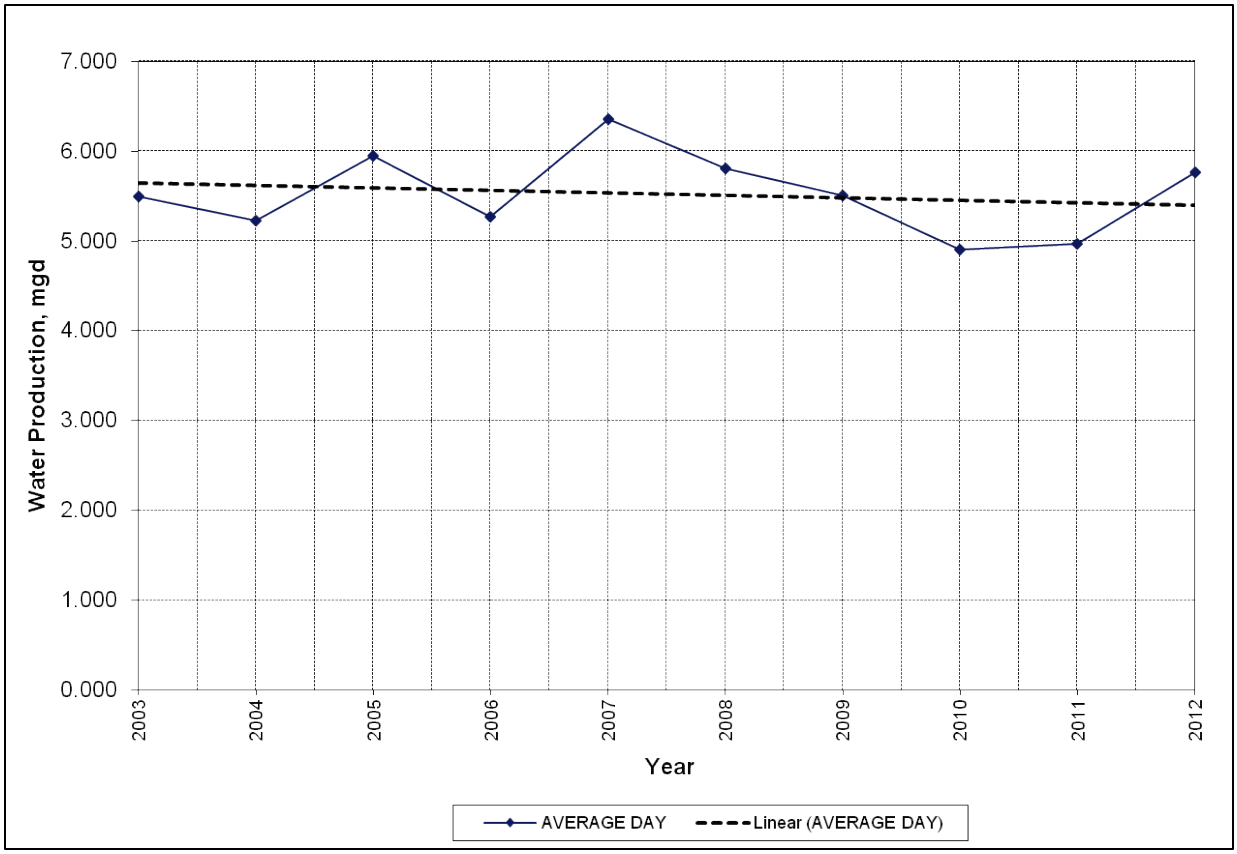


Figure 1 – Historical Pumpage for Past 10 Years

The per capita pumpage averaged 135.8 gpcd for the past 10 years. Because any unaccounted-for water is also included in the per capita figures, they are called per capita pumpage instead of per capita usage or demand.

The per capita figures presented in Table 1 and Figure 2 include water used in not only residential usage, but also other usage such as commercial and industrial. Figure 2 shows that the trend in estimated per capita pumpage has been downward over the past 10 years.

Because the per capita pumpage usages include commercial and industrial pumpages, major increases or decreases in industrial or commercial pumpage will affect the calculated per capita pumpage figures. Using past per capita pumpage figures to project future demands also assumes that growth in future commercial and industrial water pumpage will follow trends similar to the residential population trends.

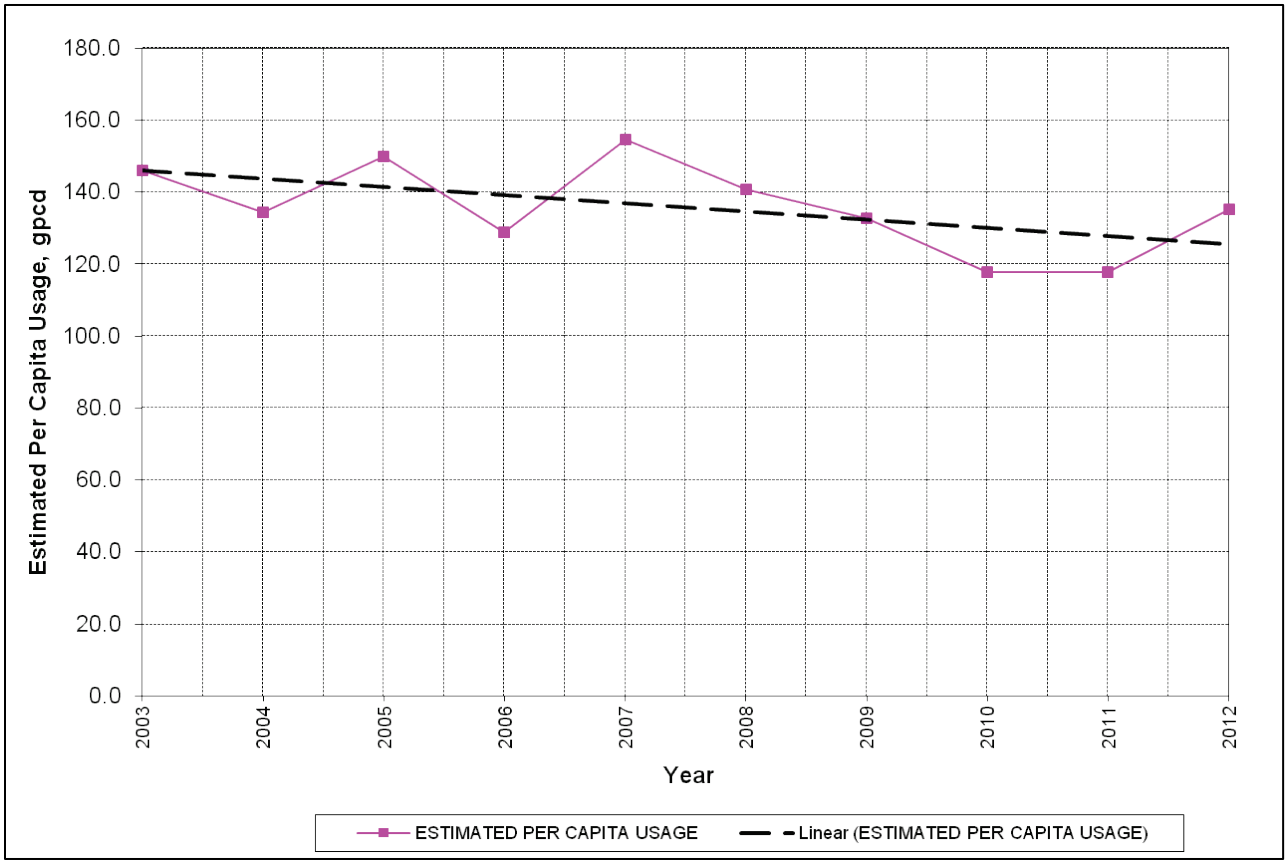


Figure 2 – Per Capita Usage Trend for Past 10 Years

3.2 LARGEST WATER USERS

Table 2 shows the largest water users in the water system for 2012. The total volume of water used by the largest ten users equates to approximately 7.66% of the total volume of water pumped.

Table 2 – Summary of Largest Water Users

No.	Customer	Year 2012 Average Water Usage (gpd)	Percentage of 2012 Pumpage
1	Edward Rose Apartments	129,545	2.24%
2	Medallion Management Apartments	58,877	1.02%
3	Stryker	48,384	0.84%
4	Crossroads Mall	47,041	0.81%
5	Greenspire Apartments	39,838	0.69%
6	Harvard Courtyards	28,685	0.50%
7	TenderCare Nursing Home	25,123	0.43%
8	Timberwood Crossing Apartments	23,175	0.40%
9	Mann - Hummel	22,986	0.40%
10	Charles River Breeding Labs	19,041	0.33%
	Total	442,696	7.66%

gpd = gallons per day

3.3 WATER ACCOUNTABILITY

Comparing the volume of water pumped with the volume of water billed is helpful in evaluating system accountability. The discrepancy between these values is due to lost or unaccounted-for water. Water distribution systems normally “lose” water due to leaks, illegal water usage, meter errors, flushing, firefighting, or other reasons.

The City does not currently have an accurate way to gauge unaccounted water due to issues with their billing software. The City is currently evaluating billing software changes which will allow for better tracking of unaccounted water.

In general, percentages of lost water in the 5 to 15% range are typical and many systems strive to reduce their percentage of unaccounted water to less than 10%.

The per capita usage and demand projections will be based on pumpage instead of usage plus unaccounted-for water.

3.4 MAXIMUM DAY DEMAND AND PEAKING FACTORS

The maximum day demand for a given year is useful for calculating system peaking factors and sizing pumping and storage facilities. Figure 3 shows maximum day pumpage for the past ten years. Average day pumpage has also been shown in Figure 3 for comparison.

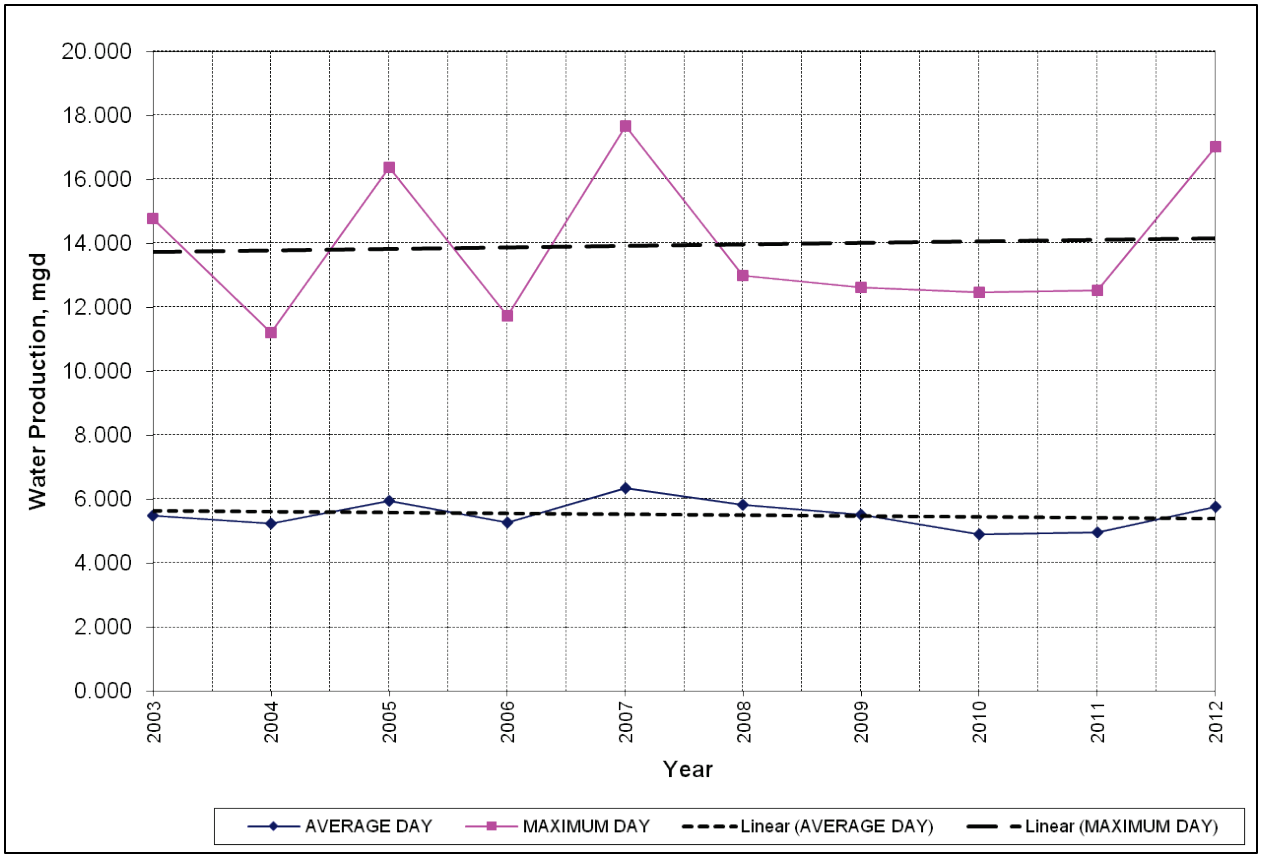


Figure 3 – Maximum and Average Day Pumpage for Past 10 Years

The trend line in Figure 3 illustrates that maximum day demands have trended upward over the past 10 years. Table 3 presents the average day and maximum day pumpage data used to generate Figure 3.

Table 3 - Maximum Day to Average Day Peaking Factors

Year	Average Day Pumpage (mgd)	Maximum Day Pumpage (mgd)	Calculated Peaking Factor
2003	5.495	14.780	2.690
2004	5.230	11.220	2.145
2005	5.950	16.380	2.753
2006	5.273	11.740	2.226
2007	6.359	17.670	2.779
2008	5.812	13.001	2.237
2009	5.509	12.630	2.293
2010	4.903	12.460	2.541
2011	4.967	12.530	2.522
2012	5.764	17.030	2.955
Average	5.526	13.94	2.514
Maximum	6.359	17.67	2.955
Minimum	4.903	11.22	2.145

The maximum day peaking factor is calculated by dividing the highest daily pumpage in a given year by the annual average pumpage for that year. The maximum day peaking factor for the City water system ranged from 2.145 to 2.955 and averaged 2.514 between 2003 and 2012. Figure 4 presents a graphical representation of the trend in peaking factors over the period. The overall trend in peaking factors over the past 10 years has been slightly increasing. While the average day pumpage rates have followed a relatively steady trend over the past 10 years, maximum day demands have varied more erratically. This has resulted in a relatively wide variation in peaking factors.

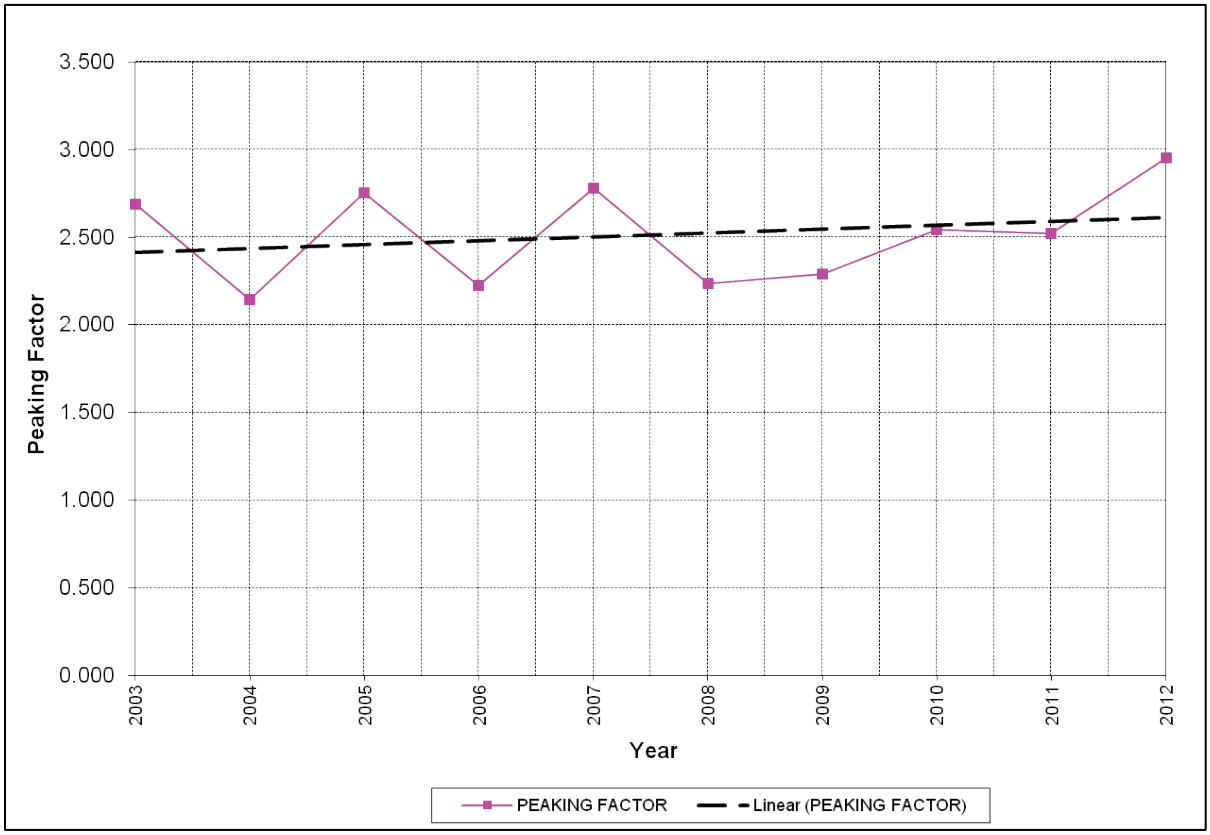


Figure 4 – Historical Peaking Factors for Past 10 Years

3.5 PEAK HOUR DEMAND AND PEAKING FACTORS

Throughout a 24-hour period, system demands will vary considerably. The water system must be capable of meeting short-term high demands. To properly evaluate and design water supply, storage, and distribution facilities, it is helpful to know the water system's peak hour demand.

In the City's system, peak hour demands are supplied by a combination of pumping capacity and feed from elevated storage. Detailed information on the peak hour flow rates was not readily available. For system analyses and hydraulic modeling, a ratio of 1.50 has been assumed for the ratio of the peak hour to maximum day demands. Other previous studies for the City had used peaking factors ranging from 1.50 to 1.53.

4.0 WATER DEMAND PROJECTIONS

Water demand projections are typically based on changes in service population. Historical population data and future population projections through 2030 were presented in the *City of Portage 2008 Comprehensive Plan* (City of Portage Planning Commission). Population projections for 2035 were extrapolated from the available projections through 2030. Table 4 summarizes historical population data and projections. The assumption was made that the percentage of the population served would not exceed 90%.

Table 4 – Historical Population Data and Future Population Projections

Year	U.S. Census Data	Comprehensive Plan Projection	Percentage Increase over Period	Percentage of Population Served	Population Served
1960	20,181	--	--	--	--
1970	33,590	--	66.4%	--	--
1980	38,157	--	13.6%	--	--
1990	41,042	--	7.6%	52.0%	21,342
2000	44,897	--	9.4%	75.8%	34,009
2010	46,292	--	3.1%	90.0%	41,663
2020	--	51,133	10.5%	90.0%	46,020
2030	--	55,737	9.0%	90.0%	50,163
2035	--	58,021	4.1%*	90.0%	52,219

*2035 percentage increase is for a 5-year period vs. 10-year periods for other years.

Figure 5 provides a graphical representation of the population data.

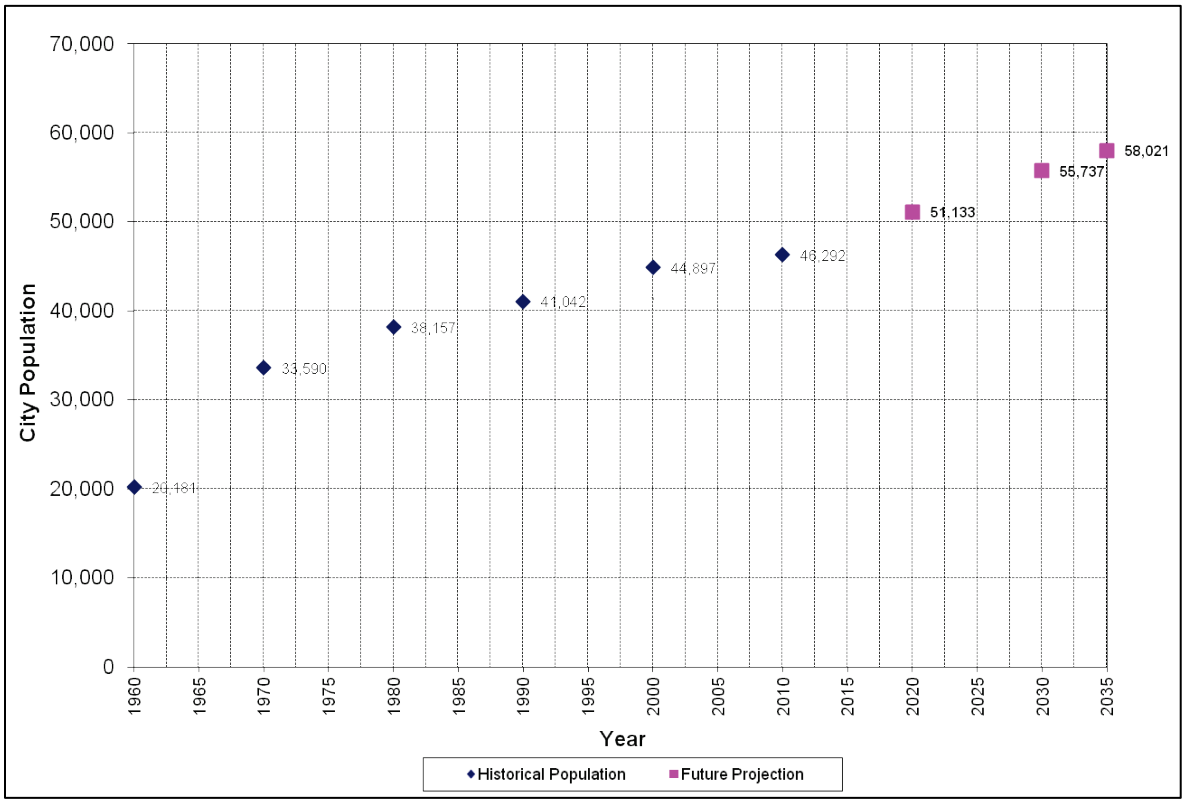


Figure 5 – Historical Population Data and Future Projections

Future water use projections were developed based on an analysis of historical data and future population projections. Because data on unaccounted-for water was not considered to be accurate, the projected numbers are for pumpage that includes both projected demand and unaccounted water. Pumpage projections were developed by multiplying the projected population by the anticipated per capita demand pumpage. As historical data shows, per capita pumpage varies from year to year. In addition, the maximum to average day peaking factor also varies significantly from year to year. To better understand the variation in per capita pumpage and peaking factors, a standard deviation analysis of historical data was completed, and is summarized in Table 5.

Table 5 – Standard Deviation Analysis

Year	Calculated Peaking Factor	Average Per Capita Per Day Pumpage
2003	2.690	146.0
2004	2.145	134.5
2005	2.753	150.0
2006	2.226	128.8
2007	2.779	154.7
2008	2.237	140.7
2009	2.293	132.8
2010	2.541	117.7
2011	2.522	117.8
2012	2.955	135.2
Average	2.514	135.8
Maximum	2.955	154.7
Minimum	2.145	117.7
Std Dev, σ	0.264	11.8
AVG + 1.5σ	2.910	153.5
AVG – 1.5σ	2.118	118.1

Based on the values in Table 6, future pumpage projections were developed. Values equal to the average ± 1.5 times the standard deviation (σ) were used. This equates to a greater than 90% level of confidence that the actual value will be within the given range. Figure 6 summarizes the 5, 10, and 20-year pumpage projections for 2018, 2023, 2033. Projected ranges have been defined for average and maximum days. The trend in pumpage is generally expected to follow the line in the middle of each range that was based on historical averages. However, for any given year the pumpage is expected to be as high as the maximum end of the range or as low as the minimum end of the range. For example, for the year 2023, the maximum day pumpage is expected to be somewhere between 15.45 and 21.23 mgd.

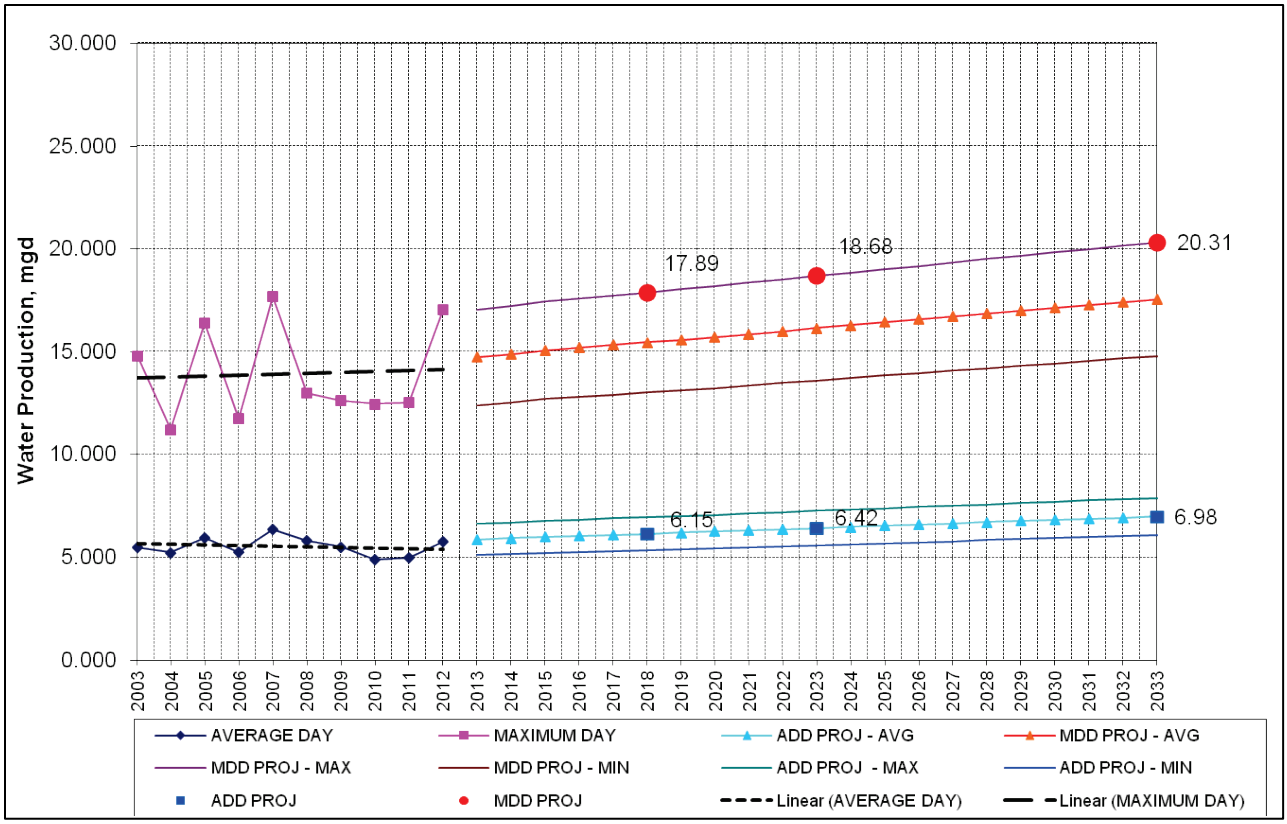


Figure 6 – Historical Pumpage for Past 10 Years with Future Projections

For sizing of water system facilities, it is recommended that the upper limit of the maximum day demand range and the average of the average day demand range be used as the demand projections. Based on this convention, the projected demands for 2018, 2023, and 2033 are presented in Table 6.

Table 6 - City of Portage Pumpage Projections

Year	Average Day Pumpage (mgd)	Maximum Day Pumpage (mgd)	Average to Maximum Day Peaking Factor	Peak Hour Pumpage (equivalent mgd)	Maximum Day to Peak Hour Peaking Factor
2018	6.15	17.89	2.91	26.83	1.50
2023	6.42	18.68	2.91	28.03	1.50
2033	6.98	20.31	2.91	30.47	1.50

5.0 WATER SUPPLY

5.1 SUPPLY CAPACITY

The City water supply consists of 20 supply wells, dispersed across the distribution system. Garden Lane Well 4 was added in 2009. A treatment plant was added to treat water from Garden Lane Wells 1 through 4. Lexington Green Wells 1 and 2 were removed from the distribution system in 2011. These system modifications were implemented to address concerns with arsenic levels. Table 7 summarizes data obtained from the City on the supply of each well.

Table 7 – Well Capacity Summary

No.	Well Name	Well Screen Capacity (mgd)	Well Pump Capacity (mgd)	2011 Well Pump Testing Capacity (mgd)
1	Amberly 1	1.35	1.44	1.44
2	Amberly 2	0.86	0.58	0.49
3	Garden Lane 1	1.36	1.22	1.44
4	Garden Lane 2	1.94	1.73	2.15
5	Garden Lane 3	1.30	1.30	0.86
6	Garden Lane 4	1.94	1.73	1.76
7	Garden Lane 5	1.30	0.94	1.15
8	Milham 1	1.37	1.22	0.88
9	Milham 2	2.07	1.44	1.44
10	Milham 3	2.72	1.44	1.07
11	Pershing 1	1.08	0.86	0.94
12	Pershing 2	0.81	0.86	0.95
13	Portage Creek	2.45	1.73	1.93
14	Rolling Hills	1.40	1.44	1.34
15	Shuman 1	1.58	1.44	1.44
16	Shuman 2	7.20	5.76	7.32
17	Wedgewood	0.86	0.58	0.63
18	Westfield 1	1.14	0.94	0.86
19	Westfield 2	0.84	0.79	0.96
20	Winterforest	0.81	1.15	0.94
	Total Capacity	34.38	28.58	30.00
	Firm Capacity			23.79

The well firm capacity is calculated with the largest supply well out of service. For the City, this criterion actually would exclude Garden Lane Wells 1 through 4, which all go through the treatment plant with no available bypass. If the treatment plant were to be out of service, Garden Lane Wells 1 through 4 would be out of service. Additionally, Winterforest and Portage Creek Wells are planned to be taken out of service in the future due to elevated arsenic concentrations. With these two wells out of service, the firm capacity is reduced to 20.92 mgd.

In accordance with the Michigan Safe Drinking Water Act, Type 1 water systems are required to have firm supply capacity sufficient to meet the maximum day demand. The current firm well capacity exceeds the projected 2013 maximum day demand and projected 20 year maximum day demand, even with Winterforest and Portage Creek wells out of service. Table 9 shows the percentage of well firm capacity. The Michigan Department of Environmental Quality often recommends additional capacity be considered when demands exceed 80% of the firm capacity, which is projected to be the case in the City in the next 10 to 20 years, and is currently the case if Winterforest and Portage Creek Wells are taken out of service.

Table 8 – Supply Capacity and Demand Comparison

Year	Maximum Day Demand (mgd)	Firm Supply Capacity (mgd)	Percent of Firm Capacity	Firm Supply Capacity with Winterforest and Portage Creek Out of Service (mgd)	Percent of Firm Capacity with Winterforest and Portage Creek Out of Service (mgd)
2013	17.05	23.79	71.7%	20.92	81.5%
2018	17.89	23.79	75.2%	20.92	85.5%
2023	18.68	23.79	78.5%	20.92	89.3%
2033	20.31	23.79	85.4%	20.92	97.1%

The data provided in Tables 7 and 8 should be considered to be approximate estimates of available well capacity and the additional capacity that is required. Screen velocity and pump capacity are two important factors in determining the capacity of a well pump. Safe aquifer yield is another important factor in determining well capacity. Safe aquifer yield depends on well drawdown, specific capacity, and the impact of wells on each other.

Additional supply capacity will eventually be required. Alternatives to increase supply capacity should be pursued now to increase capacity within the next few years, particularly if Winterforest and Portage Creek Wells are taken out of service. The City has already completed some preliminary investigations into new well sites. While there are several potential sites that may have adequate water production, preliminary data showed that iron may be present in undesirable concentrations at each of these sites.

5.1.1 INCREASE PUMPING CAPACITY AT EXISTING WELLS

One alternative that could potentially increase water supply would be the installation of larger pumps at some wells. As Table 8 showed, several wells in the system have well screen capacities that significantly exceed pumping capacities. It may be possible to replace some well pumps with larger pumps to gain additional capacity from existing wells. This could provide a cost effective means for increasing system firm capacity. Table 9 shows several wells in the system that may have additional capacity available.

Table 9 – Potential Increase in Capacity of Existing Wells

Well Name	Base Capacity (mgd)	Base Capacity (gpm)	Potential Additional Capacity (mgd)	Potential Additional Capacity (gpm)
Milham 2	1.44	1,000	0.63	437
Milham 3	1.44	1,000	1.28	889
Portage Creek	1.73	1,201	0.72	500
Shuman 2	5.76	4,000	1.44	1,000
Total			4.07	2,826

Additional study would be required to determine if it is feasible or recommendable to replace any of the well pumps in the system. Safe aquifer yield would need to be evaluated to determine if any wells could have their capacity increased.

5.2 WATER TREATMENT

A new 5.76 mgd water treatment facility was constructed at the Garden Lane Well field. The treatment facility removes arsenic, iron, and manganese from water pumped from Garden Lane Wells 1 through 4. Garden Lane Well 4 was newly installed in 2009 as part of the water treatment facility project.

Additionally, Lexington Green Wells 1 and 2 were abandoned in 2011 due to their elevated arsenic levels. Portage Creek and Winterforest Wells also have elevated arsenic levels and are planned to be removed from the system.

6.0 WATER STORAGE

The City has two elevated storage tanks serving a single pressure district, with a total storage volume of 2,250,000 gallons. The Haverhill Park Tank is a 750,000-gallon tank constructed in 1962. The Haverhill Park Tank has an overflow elevation of 1,034 feet (U.S. Geological Survey [USGS]). The Pineview Tank is a 1,500,000-gallon composite tank constructed in 1991. The Pineview Tank has an overflow elevation of 1,035 feet (USGS).

6.1 STORAGE CAPACITY

Table 11 shows the percentage of projected maximum day demands available in storage. One common guideline used to evaluate elevated storage capacity is that a system should have storage capacity equal to 25% of the maximum day demand.

Table 10 – Percentage of Demand Available in Storage

Year	Projected Maximum Day Demand (mgd)	Maximum Available Storage Volume (MG)	Maximum Storage Volume as Percentage of Maximum Day Demand	Typical Available Storage Volume Tanks ¾ Full
2013	17.05	2.25	13.2%	1.69
2018	17.89	2.25	12.6%	1.69
2023	18.68	2.25	12.0%	1.69
2033	20.31	2.25	11.1%	1.69

The percentages shown in Table 10 are significantly below the 25% guideline. To meet the 25% guideline for the projected 2033 demands, an additional 2.83 MG of storage would be required. The 25% guideline is not a regulatory requirement, and the City water system currently has additional built-in reliability due to the distribution of the existing supply wells across the system.

The City plans to replace the Haverhill Tank with a new elevated storage tank in the immediate future. Several options are being considered for the new tank. The modeling of the tank replacement is discussed in Section 8.

6.2 ELEVATED STORAGE TANK INSPECTIONS

Both elevated storage tanks were recently inspected. The Pineview Tank was inspected in 2010 by Utility Service Co., Inc. and was found in general to be in good condition and have deterioration consistent with its age. The inspection report recommended that the tank be recoated both inside and out. Exterior tank coating was completed in 2012.

A tank inspection report for the Haverhill Tank was not available. The interior of the tank was re-coated in 2009, based on the recommendation of the most recent inspection. As noted, the Haverhill Tank is scheduled for replacement in the next few years.

It is generally recommended that tank inspections be repeated every five years to identify and remedy any problems before they become severe.

7.0 WATER QUALITY

In addition to supplying water in adequate volumes to meet system demands, the quality of water supplied also needs to be considered. The City has had elevated arsenic levels in the past, as was detailed in the previous reliability study. A water treatment plant was built to remove arsenic, iron, and manganese from Garden Lane Wells 1 through 4. Additionally, the Lexington Green Wells 1 and 2 were removed from service to address elevated arsenic levels.

Iron and manganese are aesthetic concerns for the water quality in Portage. Most of the wells in the system have iron levels that exceed the secondary drinking water standard of 0.3 milligrams per liter (mg/L) for iron. The maximum contaminant level (MCL) for arsenic is 0.01 mg/L. The Winterforest Well routinely exceeds the arsenic standard; the City plans to remove this well from service. Water at the treatment plant achieved arsenic concentrations below the MCL for all sampling events reported from 2010 through 2012.

Table 11 summarizes available data on iron and arsenic concentrations obtained from the City.

Table 11 - Supply Well Iron and Arsenic Data

Supply Well	Iron (mg/L)				Arsenic* (mg/L)			
	2009	2010	2011	2012	2009	2010	2011	2012
Amberly 1	ND	ND	ND	ND	--	--	ND	--
Amberly 2		--	--	--	--	--	--	--
Garden Lane 1	1.5	--	--	--	0.019	0.013	--	--
Garden Lane 2	1	--	--	--	0.029	0.024	--	--
Garden Lane 3	0.9	--	--	--	--	0.007	--	--
Garden Lane 4	--	--	--	--	--	0.008	--	--
Garden Lane 5	0.6	0.4	0.4	0.5	--	--	ND	--
Milham 1	--	--	--	--	--	--	--	--
Milham 2	0.4	0.3	0.2	0.2	--	--	0.004	--
Milham 3	--	--	--	--	--	--	--	--
Pershing 1	0.3	--	0.2	0.2	--	--	0.004	--
Pershing 2	--	ND	--	--	--	--	--	--
Portage Creek	0.9	0.8	0.6	0.6	--	--	0.009	0.009
Rolling Hills	0.7	0.8	0.7	0.9	--	--	0.004	--
Shuman 1	1.3	1.2	0.8	0.8	--	--	ND	--
Shuman 2	--	--	--	--	--	--	--	--
Wedgewood	ND	0.1	ND	ND	--	--	ND	--
Westfield 1	0.4	0.6	0.3	0.4	--	--	0.002	--
Westfield 2	--	0.3		--	--	--	--	--
Winterforest	0.2	0.3	--	0.1	0.04	0.016	0.024	0.024
Water Treatment Plant	--	ND	ND	ND	--	0.002	0.007	0.006

*Reported arsenic concentration is the maximum value of the quarterly sampling events
ND - not detected
-- - not sampled

8.0 WATER DISTRIBUTION SYSTEM

A detailed evaluation of the City water distribution system was completed as part of the Water System Reliability Study. The evaluation included updating and calibrating the City's existing water system hydraulic model. The updated water system hydraulic model was then used to evaluate the capability of the system to reliably meet system demands and provide fire flows at adequate pressures. Recommendations for system improvements were developed based on results of the hydraulic modeling and other analyses of the water system.

8.1 WATER DISTRIBUTION SYSTEM PIPE INVENTORY

The City has a general plan showing all water system facilities and the water service area as part of their GIS. The plan is up to date and well-maintained. The future service area is expected to cover the same area as the existing area, with new customers being primarily located within the existing service area boundaries. There are portions of the City of Portage that are served by City of Kalamazoo. These areas are located north of I-94 and west of US-131 and are anticipated to continue to be served by Kalamazoo into the foreseeable future.

Tables 12, 13, and 14 list the water mains by size, material, and age.

Table 12 – Pipe Inventory by Size

Nominal Pipe Diameter (inches)	Total Length (ft)	Percent of Total
not specified	8,126	0.58%
4	7,898	0.56%
6	219,833	15.62%
8	681,236	48.40%
10	44,172	3.14%
12	297,957	21.17%
14	561	0.04%
16	109,949	7.81%
20	37,812	2.69%
Total	1,407,544	100.00%

Table 13 – Pipe Inventory by Material

Pipe Material	Total Length (ft)	Percent of Total
not specified/unknown	10,585	0.75%
Ductile Iron	1,068,539	75.92%
Cast Iron	327,859	23.29%
HDPE	561	0.04%
Total	1,407,544	100.00%

Table 14 – Pipe Inventory by Age

Year Installed	Total Length (ft)	Percent of Total
not specified	1,706	0.12%
1951-1960	73,847	5.25%
1961-1970	277,792	19.74%
1971-1980	259,514	18.44%
1981-1990	263,774	18.74%
1991-2000	284,166	20.19%
2001-2010	235,526	16.73%
2011-present	11,219	0.80%
Total	1,407,544	100.00%

8.2 HYDRAULIC MODEL UPDATES

A hydraulic model of the water distribution system was updated and used to identify deficiencies in the current system and to assist in planning for future development of the system. Haestad Methods WaterCAD® V8i software was used to create the model of the distribution system.

The City had an existing hydraulic model that was developed as part of the previous reliability study. This model had also been calibrated as part of the previous study. Updates were made to the existing model to reflect water distribution system improvements completed since its development. The model was then re-calibrated as part of this study.

Average day demands were allocated by inputting actual average demands for the ten largest users based on annual billing records provided by the City for 2012, and then distributing the remainder of the demand evenly across the system. Maximum day and peak hour demands were developed by scaling the average day demands up based on the peaking factors described in previous sections of this report.

8.3 HYDRAULIC MODEL CALIBRATION

Model calibration is essential for model results to be accepted with a high degree of confidence. Model calibration can also reveal areas where the system is in better or worse condition than expected. The calibration process involves obtaining field data and adjusting the model to simulate those field conditions. Hydrant flow testing was the primary means of collecting field data.

8.3.1 HYDRANT FLOW TESTING

Hydrant flow test locations were chosen to match with some of the locations tested as part of the previous reliability study. This allowed the results to easily be compared to previous results and specifically identify if any of these locations had seen degradation in the past 8 years. In choosing the hydrant flow test locations originally, the age, diameter, and material of the pipes in the system were considered. Locations were chosen to sample a wide variety of different age and material pipe. In total, 8 flow tests were completed, with five individual fire hydrants used in each test. For each test, one hydrant was flowed, while residual pressures were observed at four adjacent hydrants.

8.3.2 MODEL CALIBRATION ADJUSTMENTS

System operational data was also recorded during the flow tests, including the levels in the elevated tanks, and which wells were operating. The model was then adjusted to reflect actual system operation. Hazen-Williams C-factors were adjusted to match field data. Several iterations with different C-factor values were completed to calibrate the model.

Static pressures in the model were within 2 psi of field data or better for all 40 locations. Modeled residual pressures were within 5 psi of field data or better, with the exception of 1 of 40 locations, which had discrepancies of 5.7 psi.

The accuracy of model calibration is considered to be very good. The model can be used with confidence for analyzing the system.

8.4 2013 DEMAND MODEL RUNS

The model was run at the projected 2013 and 2033 demand conditions to evaluate the performance of the system at current and future conditions. The minimum desired pressure in the system is 35 psi.

8.4.1 2013 MAXIMUM DAY DEMAND MODEL RUNS

The calibrated model was run with 2013 maximum day demand conditions (17.05 mgd) to evaluate the current distribution system. During the model runs, a combination of well pumps was selected to supply demand and keep elevated storage tank levels balanced. System pressures ranged from 42 to 83 psi during 2013 maximum day demand conditions runs.

8.4.2 2013 Peak Hour Demand Flow Model Runs

The calibrated model was run with 2013 peak hour demands (25.57 mgd). The model was run based on the system firm capacity. All well pumps were run at full capacity with the exception of the largest well, Shuman 2, being out of service. Under these conditions, both of the elevated storage tanks were draining. After one hour, the Haverhill Tank had dropped from 81.9% full to 78.1% full and was draining at a rate of 464 gpm. The Pineview Tank had dropped from 86.8% full to 83.5% full and was draining at a rate of 770 gpm. System pressures after a one-hour duration were between 41 and 83 psi. System pressures were above the recommended minimum pressure of 35 psi after one hour. Figure 7 shows pressure contours for the system after one hour of peak hour demands.

8.4.3 2013 Available Fire Flow Model Runs

An analysis of the available fire flow was also completed. The available fire flow analysis calculated the flow rate that can be withdrawn from the system at a given node in the model while maintaining a pressure of 20 psi at all other nodes in the model. This analysis was completed for each node in the system during maximum day 2013 demands (17.05 mgd). Figure 8 depicts available fire flow contours for this scenario.

The vast majority of the system had available fire flows of 1,500 gpm or much greater. There were some small areas where available fire flows were less than 1,500 gpm, and even smaller isolated areas where available fire flows were less than 1,000 gpm. The recommended fire flow depends on the type of use in the area (i.e., residential, industrial, commercial, etc.). Consistent with the previous study, the City established two flow rate goals as minimum available fire flow for all areas, except the Sprinkle Road corridor, for which fire officials would like to see 6,500 gpm. One scenario considered all areas with fire flows less than 1,000 gpm for improvements. A second scenario considered all areas with fire flows less than 1,500 gpm for improvements.

8.5 2033 DEMAND MODEL RUNS

The ability of the water distribution system to serve projected increases in demands was evaluated using the model.

8.5.1 2033 Maximum Day Demand Model Runs

The calibrated model was run at 2033 maximum day demand conditions (20.31 mgd) to evaluate the capability of the existing system to meet projected demands. Wells pumps were turned on in the model to match the demand.

The existing system pressures ranged from 43 to 82 psi during 2033 maximum day demand conditions.

8.5.2 2033 Peak Hour Demand Flow Model Runs

The calibrated model was then run at 2033 peak hour demands (30.47 mgd). The same well capacity was maintained as in the 2013 peak hour demand model runs. Under peak hour demands, both elevated storage tanks were draining rapidly. After a one hour duration, the Haverhill Tank had dropped from 81.9% full to 63.0% full and was draining at a rate of 2,020 gpm. After one hour the Pineview Tank had dropped from 86.8% full to 79.0% full and was draining at a rate of 2,053 gpm. System pressures after a one hour duration were between 38 and 80 psi. The lowest system pressures are in the vicinity of Westfield Park, west of US-131. The low pressures here are due primarily to a higher relative elevation compared with the rest of the system. System pressures were not expected to fall below the recommended minimum pressure of 35 psi. Figure 9 shows graphical results for the 2033 peak hour model run with pressure contours after a one hour duration.

8.5.3 2033 Available Fire Flow Model Runs

An analysis of the available fire flow was also evaluated for maximum day 2033 demands (20.31 mgd). The same pumps were operated as in the 2033 maximum day demand runs. The available fire flows were similar to 2013, but slightly lower at most locations. In general, the areas with low available fire flows were enlarged slightly over 2013. The results of the available fire flow analysis are presented graphically in Figure 10 using available fire flow contours.

8.6 SUMMARY OF EXISTING SYSTEM DEFICIENCIES

In general, the Portage distribution system has very few deficient areas. The system benefits from being relatively new, with a good network of large diameter transmission mains to move water across the system. Deficiencies were generally limited to small isolated areas with small diameter or dead-end mains.

8.6.1 Pressure Deficiencies

There is one area where system pressures are anticipated to be approaching deficient. On the northwest end of the system, the area in the vicinity of Westfield Park has pressures that fall below 40 psi at 2033 peak hour demands. Pressures between 35 and 40 psi are acceptable but marginal. This is primarily due to the high elevation of this area relative to the rest of the system. Maintaining pumping capacity in excess of the capacity required to meet demands would limit the rate of drain in the elevated storage tanks and keep pressures up at high elevation areas. Alternately, the Westfield area could be isolated as a separate pressure district at a higher hydraulic grade. While this would increase pressures, it would also complicate system operation.

8.6.2 FIRE FLOW DEFICIENCIES

Many areas that were called out in the previous reliability study as having deficient fire flows have been improved to bring the fire flows up to more desirable levels. This discussion includes only those remaining areas with deficient fire flows.

On the northwest end of the system, in the vicinity of Cedarcrest and Briarhill Streets, available fire flows are below 1,600 gpm. A small increase in projected demands could leave these areas with available fire flows of less than the recommended 1,500 gpm. Available fire flows in this area are limited by 6-inch and 8-inch mains serving this area.

On the northwest end of the system near Angling Road and Hollow Wood Street, there is a small area with available fire flows less than 1,500 gpm. Improvements have recently been made in this area which increased available fire flows, but the ends of two dead end mains still have available fire flows of less than 1,500 gpm.

It is important to note that many short, dead-end segments in the distribution system are not included in the skeletonized model. While these areas have not been specifically analyzed by the model, in general, these areas can be expected to have marginal available fire flows compared to areas with looped mains.

8.7 PROPOSED SYSTEM IMPROVEMENTS AND MODEL RUNS

Improvements were developed to remedy deficiencies in the portions of the existing system with fire flows below the established goals. The model was used to evaluate improvements and determine which were most effective. Hydraulic modeling for the future improvement scenarios was completed using the 2033 projected demands.

Various improvement scenarios were developed and evaluated at several demand conditions. The first set of improvement scenarios included the replacement or addition of new pipe in the system to improve fire flows in deficient areas to 1,000 gpm and 1,500 gpm. The second set of improvement scenarios included the pipe improvements in the first scenario, plus a new elevated storage tank to replace the Haverhill Tank.

8.7.1 PIPE IMPROVEMENT SCENARIOS

One pipe improvement scenario evaluated improvements to increase available fire flows to greater than 1,000 gpm in all areas included in the model. A second pipe improvement scenario evaluated improvements to increase available fire flows to greater than 1,500 gpm in all areas in the model. Table

15 provides a prioritized list of the proposed piping improvements to improve fire flows to at least 1,000 gpm in all areas.

Table 15 – Summary of Proposed 1,000 gpm Pipe Improvements

Project No.	Project Description	Pipe Diameter (inches)	Approximate Pipe Length (lft)
A-1	Portage Road, Helen to Byrd	12	900
A-2	West End - North of South Shore	8	1,000
A-3	Point-O-Woods and Point-O-Woods Ct.	8	1,000
A-4	School Road – Loop to South to Snowberry	8	200

Table 16 provides a prioritized list of the proposed piping improvements to improve fire flows to 1,500 gpm in all areas. The 1,500 gpm improvements analysis assumes that all of the 1,000 gpm improvements have already been completed. It is recommended that as a minimum, improvements should be implemented to increase fire flows to 1,000 gpm or greater in all areas.

Table 16 – Summary of Proposed 1,500 gpm Pipe Improvements

Project No.	Project Description	Pipe Diameter (inches)	Approximate Pipe Length (linear feet)
B-1	Downing Road – Westfield to Briarhill	12	1,900
B-2	Fawn Cove Road - West of Shirley Court	8	1,100
B-3	Brynmawr Road - East of Angling Road	10	1,200
B-4	Quarter Lane - South Shore to Lakeview	8	200

Figure 11 shows the location of the proposed improvements. The 1,000 gpm improvements are shown in orange and the 1,500 gpm improvements are shown in red.

8.7.2 ELEVATED STORAGE IMPROVEMENT SCENARIOS

Recommendations for additional elevated storage in the system were provided in Section 6. The minimum total storage volume recommended is 4.26 MG for 2013 demand conditions and 5.08 MG for 2033 demand conditions based on the criteria of having 25% of maximum day demand available in storage. The hydraulic model was used to evaluate a new elevated storage tank.

8.7.2.1 New Elevated Storage Tank at Haverhill Park Area

Haverhill Park is centrally located with respect to system demands and has several large-diameter pipes in its vicinity. Hydraulic modeling showed that the flow rates from the existing Haverhill Park Tank during peak hour demands were significantly higher than from the Pineview Tank. While this shows that the Haverhill Tank is well-connected to the distribution system, it also showed that the Haverhill Tank drained much more quickly than the Pineview Tank. This is due to the fact that the volume of the Haverhill Tank is

approximately half the volume of the Pineview Tank. Additional storage volume at the Haverhill Park site would equalize levels between tanks.

Model runs were completed to determine the optimal size of a new tank at the Haverhill Park Site. Modeling showed that the ideal tank volume for equalizing peak hour demands while keeping similar levels between tanks was 2.5 MG. A 2.25 MG tank was also feasible, but the differential levels between tanks increased as the tank volume decreased.

At 2033 peak hour demands, after one hour, pressure in the system ranged from 37 psi to 78 psi. The new Haverhill Tank had dropped from 88.3% full to 76.8% full and was emptying at a rate of 4,365 gpm. During this time, the Pineview Tank was emptying at a slower rate of 1,030 gpm and went from 86.8% full to 83.5% full.

The model was then run at 2033 maximum day demand conditions with a 6,500 gpm demand on Sprinkle Rd in the industrial area. After 4 hours at these conditions, pressures in the system ranged from 14 psi to 76 psi. The pressures in the surrounding area dropped below the required 20 psi minimum pressure during a fire flow. A fire flow of 6,500 gpm could only be sustained for a few minutes before system pressures dropped below 20 psi. However, a flow of 6,000 gpm could be sustained for 4 hours with acceptable system pressure. After 4 hours of a 6,000 gpm fire flow, the proposed tank dropped from 88.3% full to 51.5% full.

8.7.3 MODEL OUTPUT WITH SYSTEM IMPROVEMENTS

Figures have been provided showing model output with the recommended elevated storage and the 1,000 gpm piping improvements in place. Modeling assumes that a new 2.5 MG elevated tank at Haverhill Park will replace the existing Haverhill Tank. Figure 12 shows available fire flows with these improvements in place. Areas of the system that currently have low available fire flows would be significantly improved with the recommended improvements. Figure 13 shows system pressures at 2033 peak hour demand conditions after one hour.

One item that is important to point out is that system pressures will continue to drop from the values shown in Figure 13 after one hour, as tank levels fall. This is projected to cause pressures to drop below 35 psi in the vicinity of Westfield Park after one hour, due to the high relative elevation there. Table 17 describes the system pressures at the lowest pressure location (intersection of Westfield Avenue and 12th Street) corresponding to various tank levels. The data provided is with proposed improvements in place, at 2033 peak hour demands.

Table 17 – Minimum System Pressures in Westfield Area

Condition	Haverhill Tank Elevation (feet)	Minimum System Pressure (psi)
Tank Full	1,034	38.2
Tank 2/3 Full	1,021	34.3
Tank 1/2 Full	1,014	32.0
Tank almost empty	995	25.6

It is expected that the tanks will normally have levels maintained at 2/3 full or greater. During sustained peak hour demands which lower tank levels, pressures may fall below 35 psi in the Westfield area. Maintaining pumping capacity in excess of the capacity required to meet maximum day demands would limit the rate of drain in the elevated storage tanks and keep pressures up at high elevation areas. Alternately, the Westfield area could be isolated as a separate pressure district at a higher hydraulic grade. While this would increase pressures, it would also complicate system operation.

9.0 SUMMARY OF RECOMMENDATIONS AND COST ESTIMATES

This report has identified several recommendations to improve water service to the City. The following summarizes these recommendations, with estimated costs where available:

9.1 INCREASE WATER SUPPLY CAPACITY

Demands are projected to approach firm water supply capacity by 2033, especially if Winterforest and Portage Creek wells are taken out of service. To meet 2033 firm capacity recommendations, approximately 4.5 mgd of new well capacity would be necessary. Supply capacity could be increased by constructing new wells or replacing existing well pumps with new higher capacity pumps where aquifer and well screen conditions allow.

Detailed cost estimates for increasing water supply capacity were not estimated. The cost could vary significantly based on the location chosen for new supply wells and the number of wells required. If new, larger pumps can be installed at some existing wells, the cost of increasing supply capacity could be significantly lower.

It is recommended that the City take steps to increase supply well capacity in the near future. If new wells are to be added or larger pumps are installed at existing wells, it is recommended that additional hydrogeological study be completed to evaluate the effects on the supply aquifers, similar to analyses conducted for new wells.

9.2 ELEVATED STORAGE IMPROVEMENTS

System elevated storage capacity should be increased to a total of 5.0 MG. A new 2.5 MG tank at Haverhill Park to replace the existing Haverhill Tank is recommended. The costs presented provide allowances for contingencies and engineering.

1.	2.5 MG Haverhill Park Tank (including demolition of existing tank)	$ 5,928,000
2.	Pineview Tank Interior Coating	$ 350,000
	Elevated Storage Total	$ 6,278,000

9.3 DISTRIBUTION SYSTEM IMPROVEMENTS

The estimated costs for distribution system improvements to improve fire flows to 1,000 gpm or greater are summarized below. The numbering corresponds to numbered locations on Figure 11. Projects have been numbered in order of approximate priority. Estimated costs are based on unit costs per lineal foot.

Unit costs of $145, $150, and $160 per linear foot were used for 8-inch, 10-inch, and 12-inch main respectively. These unit costs include allowances for engineering and contingencies.

A-1	Portage Road from Helen to Byrd (900 lft of 12-inch)	$ 187,000
A-2	West End - North of South Shore (1,000 lft of 8-inch)	$ 189,000
A-3	Point-O-Woods and Point-O-Woods Ct. (1,000 lft of 8-inch)	$ 189,000
A-4	School Road – Loop South to Snowberry (200 lft of 8-inch)	$ 38,000
	Distribution System Improvements Total	$ 603,000
B-1	Downing Road – Westfield to Briarhill (1,900 lft of 12-inch)	$ 395,000
B-2	Fawn Cove Road – West of Shirley Court (1,100 lft of 8-inch)	$ 207,000
B-3	Brynmawr Road – East of Angling Road (1,200 lft of 10inch)	$ 234,500
B-4	Quarter Lane – South Shore to Lakeview (200 lft of 8-inch)	$ 38,000
	Distribution System Improvements Total	$ 874,000

The total estimated cost to complete both the 1,000 gpm and 1,500 gpm improvements is $1,477,000.

9.4 INVESTIGATE UNACCOUNTED-FOR WATER

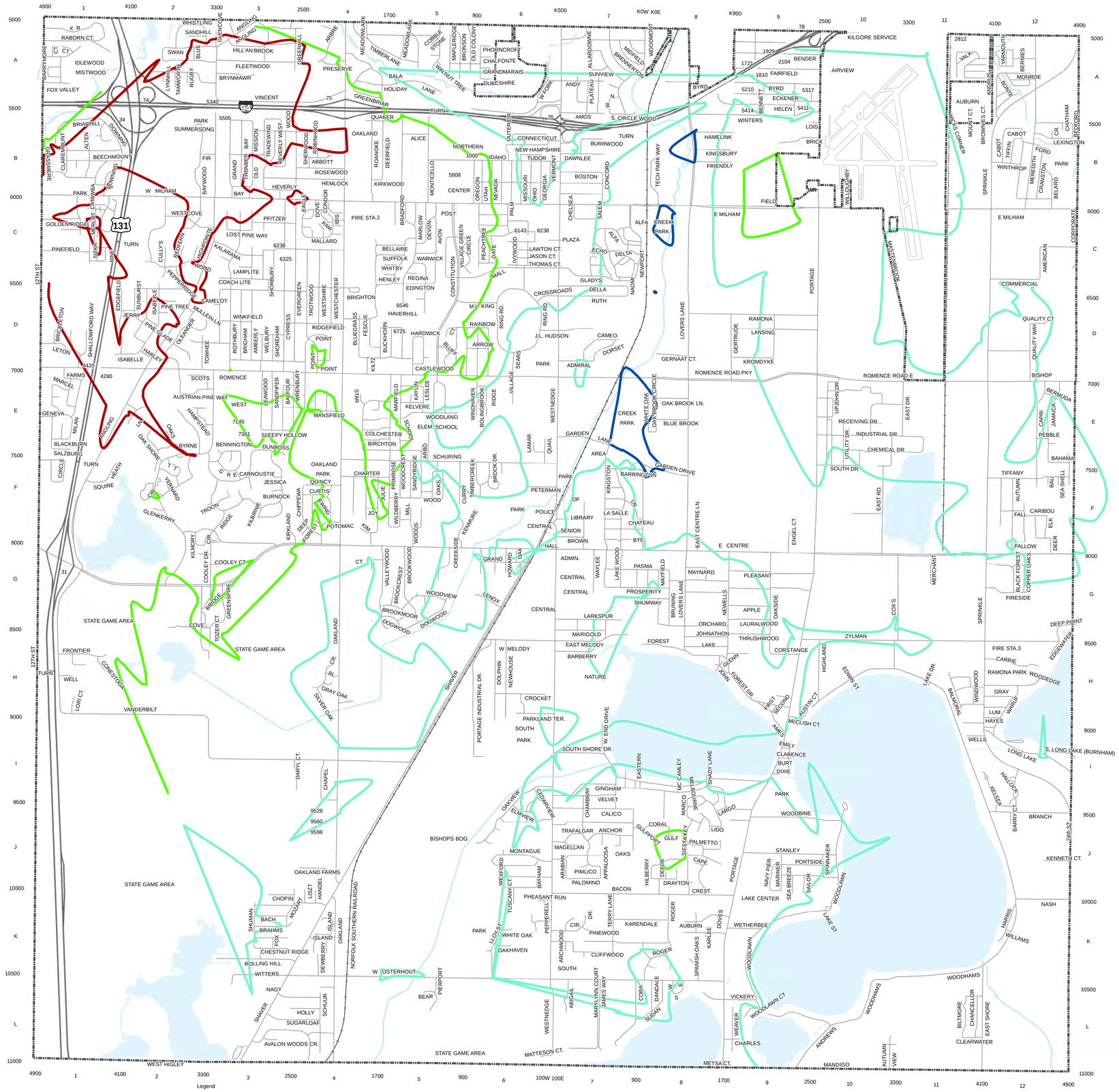
It is recommended that the percentage of unaccounted-for water be investigated in greater detail. As described in Section 3.3, there is some uncertainty as to the accuracy of the billed volumes recorded. The City should develop an accurate method of recording billed volumes for comparison with pumped volumes in future years. A cost has not been estimated for this effort.

9.5 PRIORITIZATION OF IMPROVEMENTS

It is suggested that improvements be implemented in the following order:

1. Construct 2.5 MG elevated storage tank at Haverhill Park. Complete interior coating of Pineview Tank. Construct remaining distribution system improvement projects in the approximate order listed previously.
2. Increase well supply capacity.
3. Investigate additional storage capacity.

Figures



LEGEND

Pressure Contours (psi)

- 40
- 50
- 60
- 70
- 80

NORTH

0 1,000 2,000 4,000 FEET

EXISTING WATER SYSTEM
2013 PEAK HOUR DEMAND
PRESSURE CONTOURS AFTER 1 HOUR

fr c&h

engineers
scientist
architects
constructors

fishbeck, thompson, carr & huber, inc.

City of Portage
Kalamazoo County, Michigan

Water System Reliability Study

Drawn By:	RS2
Designer:	GMV
Reviewer:	JW
Manager:	JW

Hard copy is intended to be 24"x36" when plotted. Scale(s) indicated and graphic quality may not be accurate for any other size.

PROJECT NO.
G120751

FIGURE NO.
7

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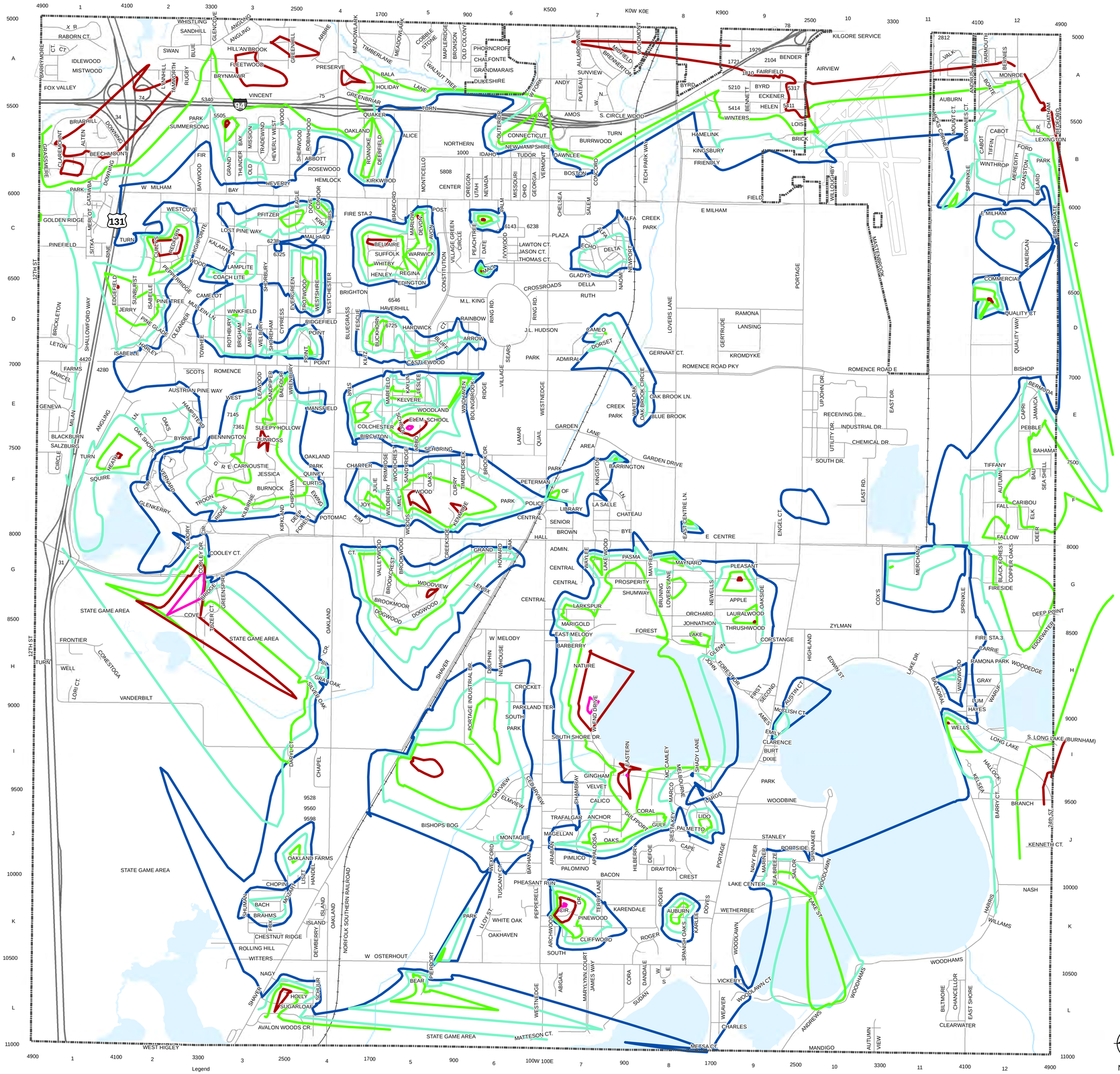

EXISTING WATER SYSTEM
2013 MAXIMUM DAY DEMAND
FIRE FLOW CONTOURS

fic&h

engineers
scientist
architects
constructors

fishbeck, thompson, carr & huber, inc.

City of Portage
Kalamazoo County, Michigan

Water System Reliability Study

Drawn By:	RS2
Designer:	GMV
Reviewer:	JW
Manager:	JW

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PROJECT NO.
G120751

FIGURE NO.
8

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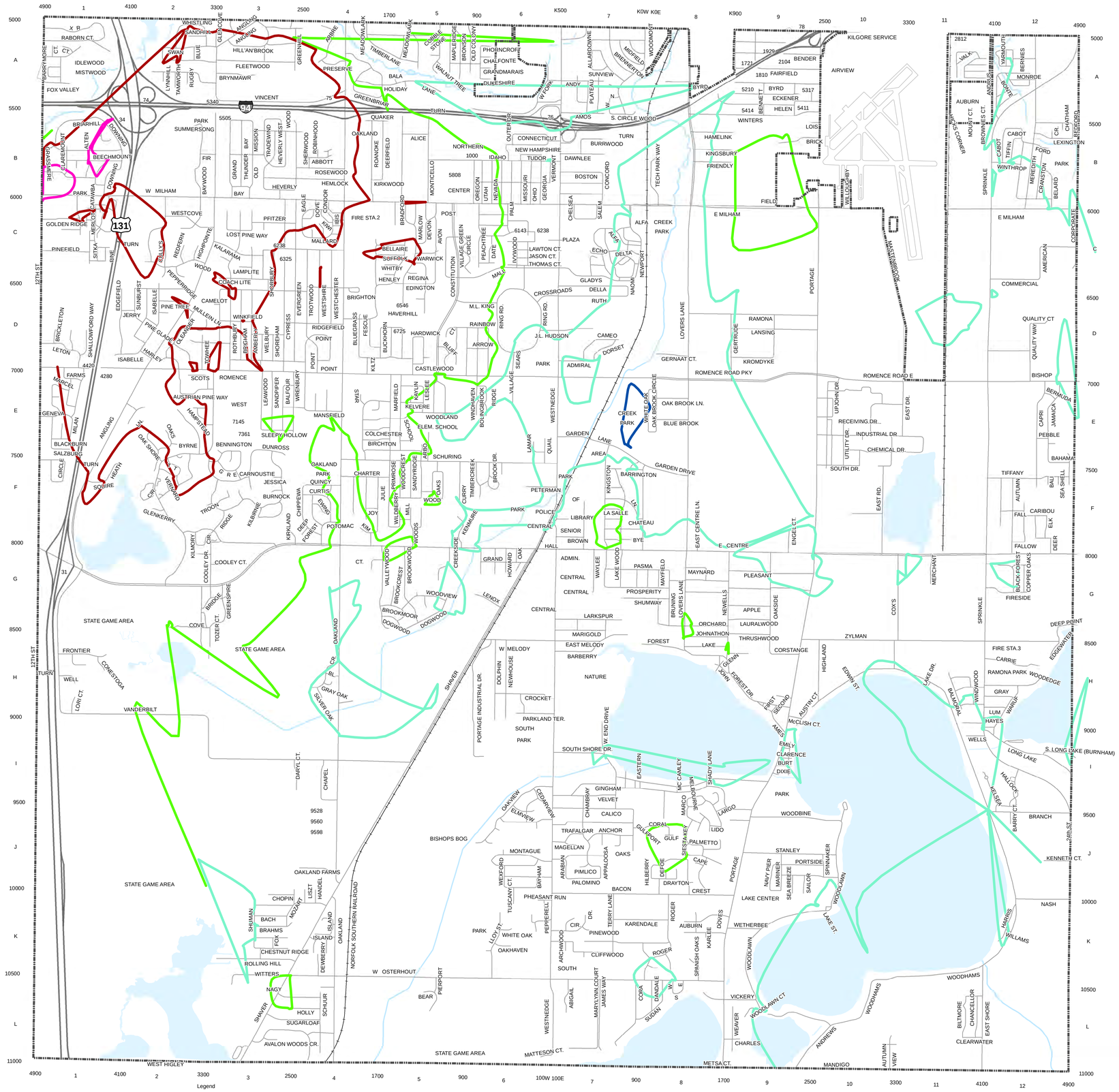

LEGEND

Pressure Contours (psi)

- 40
- 50
- 60
- 70
- 80

NORTH

0 1,000 2,000 4,000 FEET

EXISTING WATER SYSTEM
2033 PEAK HOUR DEMAND
PRESSURE CONTOURS AFTER 1 HOUR

fric&h

engineers
scientist
architects
constructors

fishbeck, thompson, carr & huber, inc.

City of Portage
Kalamazoo County, Michigan

Water System Reliability Study

Drawn By:	RS2
Designer:	GMV
Reviewer:	JW
Manager:	JW

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PROJECT NO.
G120751

FIGURE NO.
9

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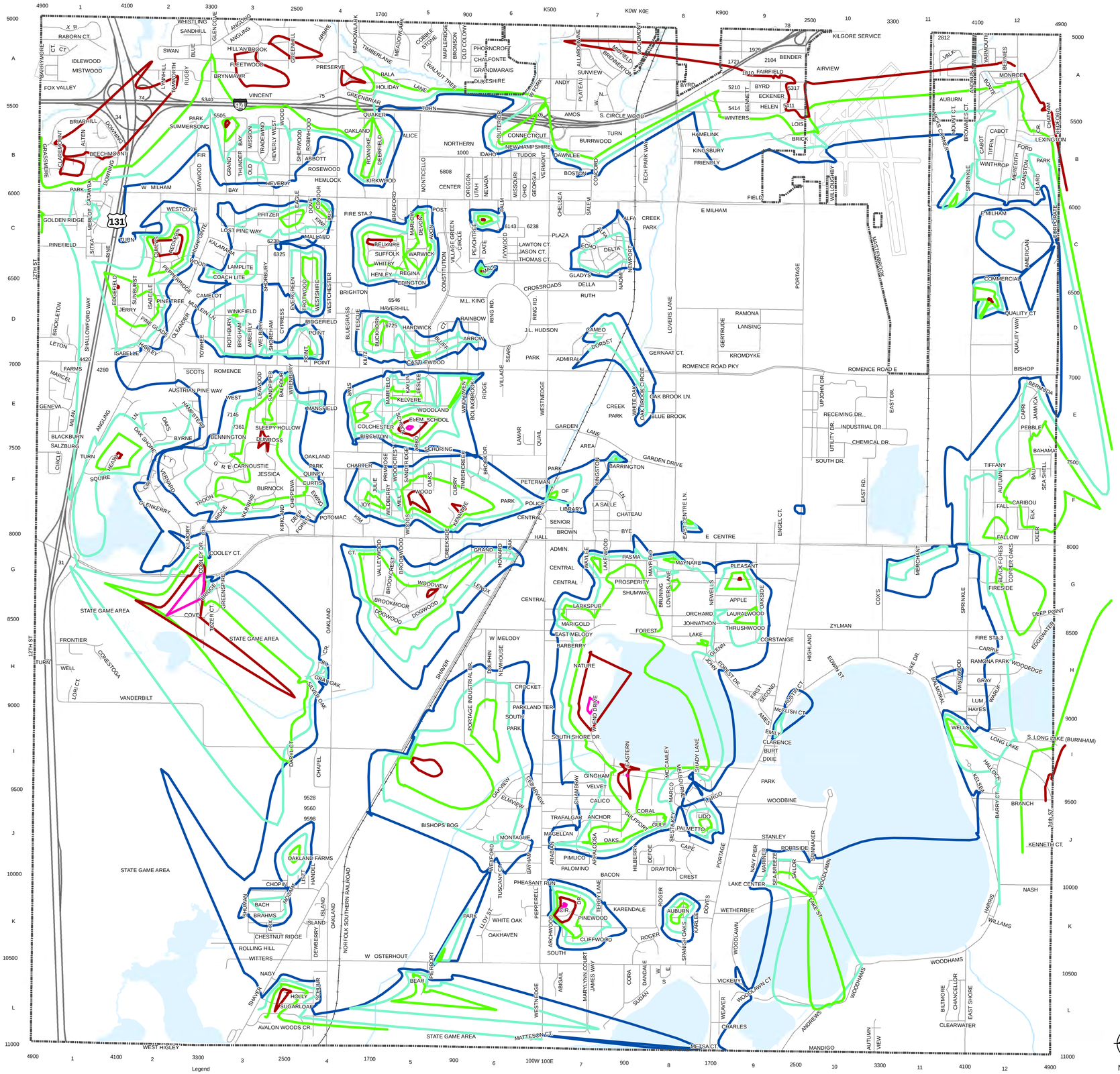

fTC&h

engineers
scientist
architects
constructors

fishbeck, thompson, carr & huber, inc.

City of Portage

Kalamazoo County, Michigan

Water System Reliability Study

Drawn By:	RS2
Designer:	GMV
Reviewer:	JW
Manager:	JW

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PROJECT NO.

G120751

FIGURE NO.

10

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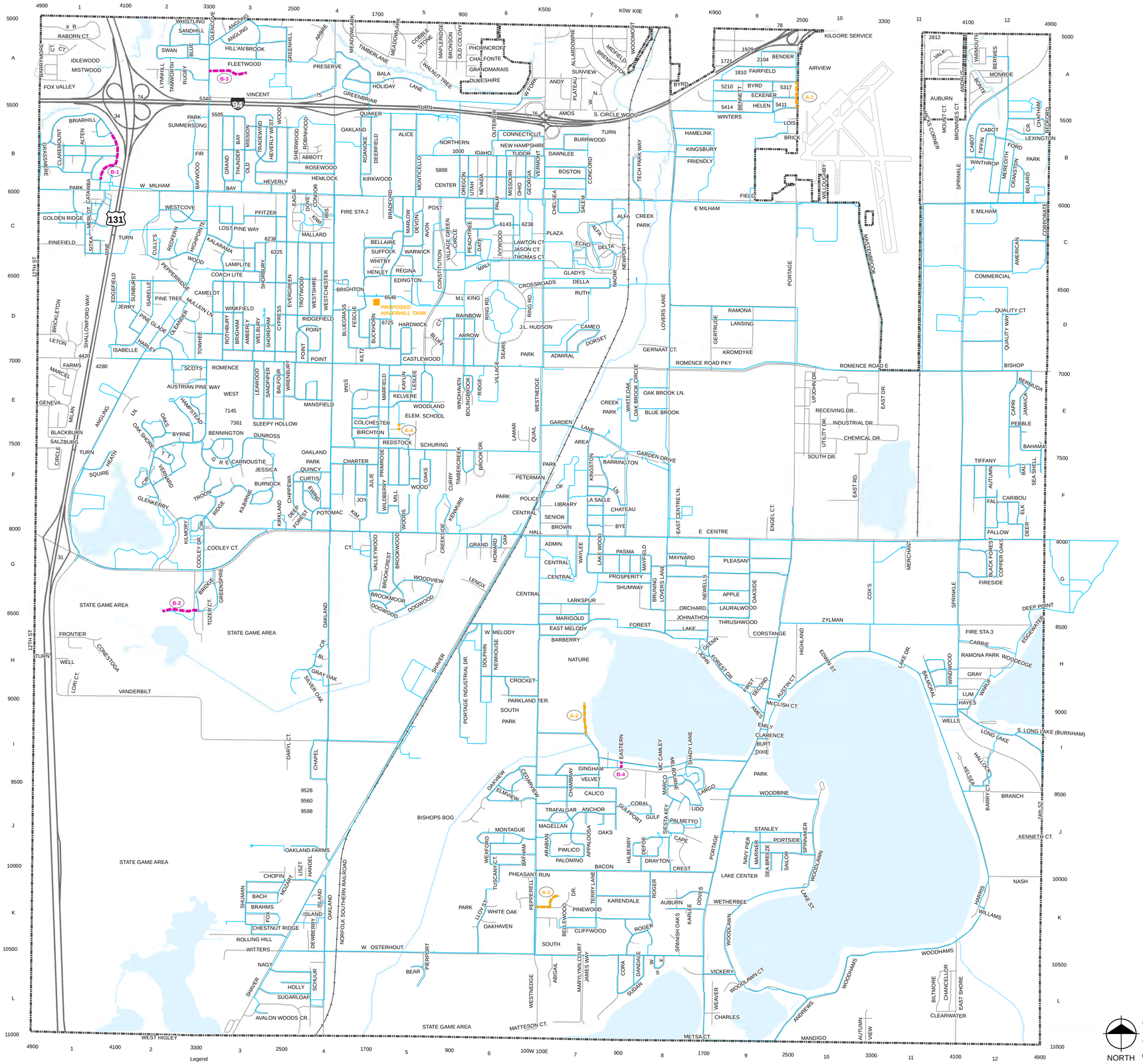

LEGEND

- Existing Modeled Water Mains
- 1000 gpm Improvements
- 1500 gpm Improvements

PROPOSED IMPROVEMENTS LOCATION MAP

NORTH

0 1,000 2,000 4,000 FEET

fric&h

engineers
scientist
architects
constructors

fishbeck, thompson, carr & huber, inc.

City of Portage

Kalamazoo County, Michigan

Water System Reliability Study

Drawn By:	RS2
Designer:	GMV
Reviewer:	JW
Manager:	JW

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PROJECT NO.

G120751

FIGURE NO.

11

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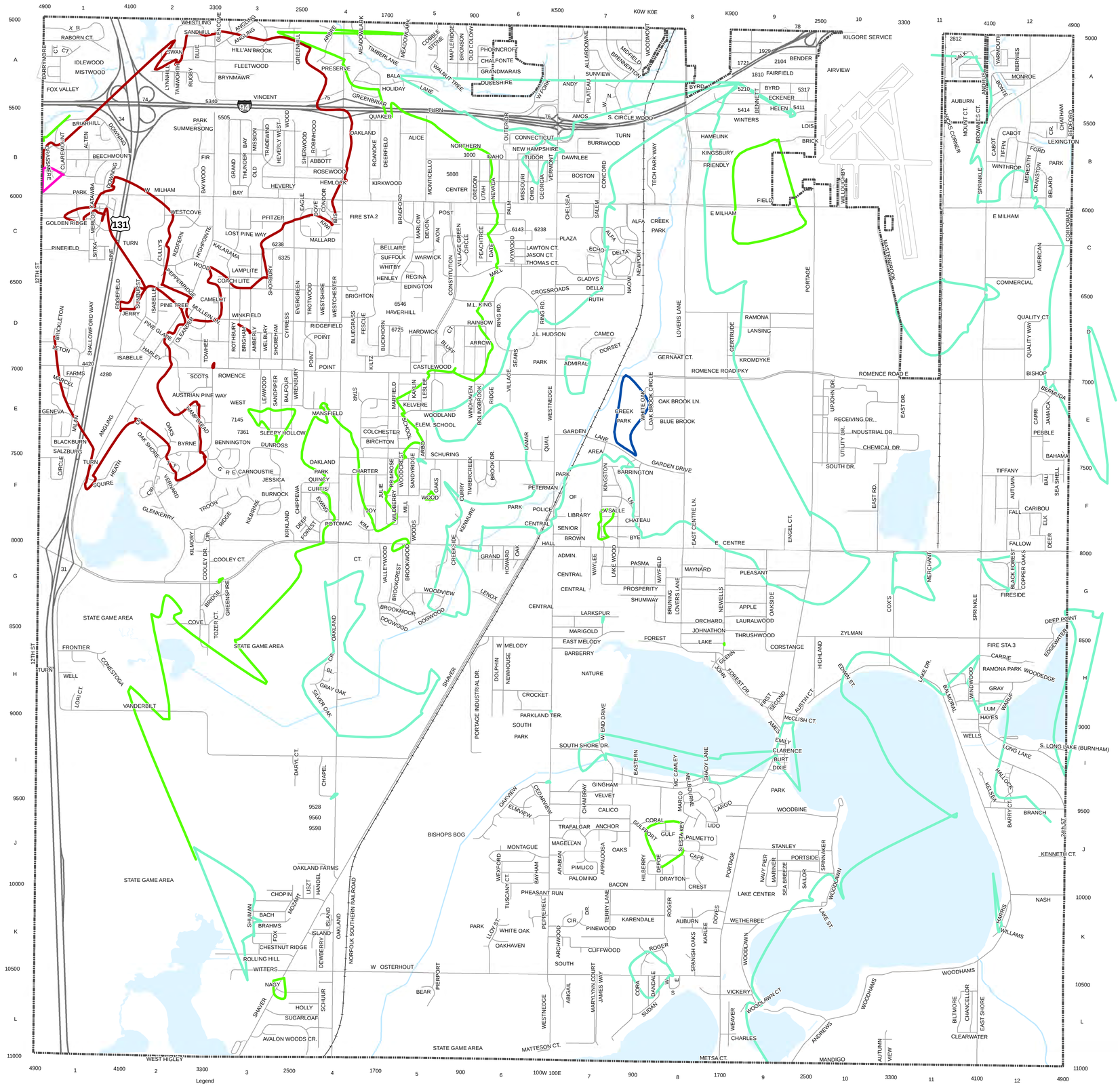
P:\GIT\INFO_2\2013\2013\UCAD\GIS\Map_Documents\Projects\Location\Map.mxd Date: 4/30/2013 7:28:16 AM User: rbg

Legend



LEGEND

Pressure Contours (psi)

- 40
- 50
- 60
- 70
- 80

IMPROVED SYSTEM
2033 PEAK HOUR DEMAND
PRESSURE CONTOURS AFTER 1 HOUR

NORTH

0 1,000 2,000 4,000 FEET

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PROJECT NO.
G120751

FIGURE NO.
12

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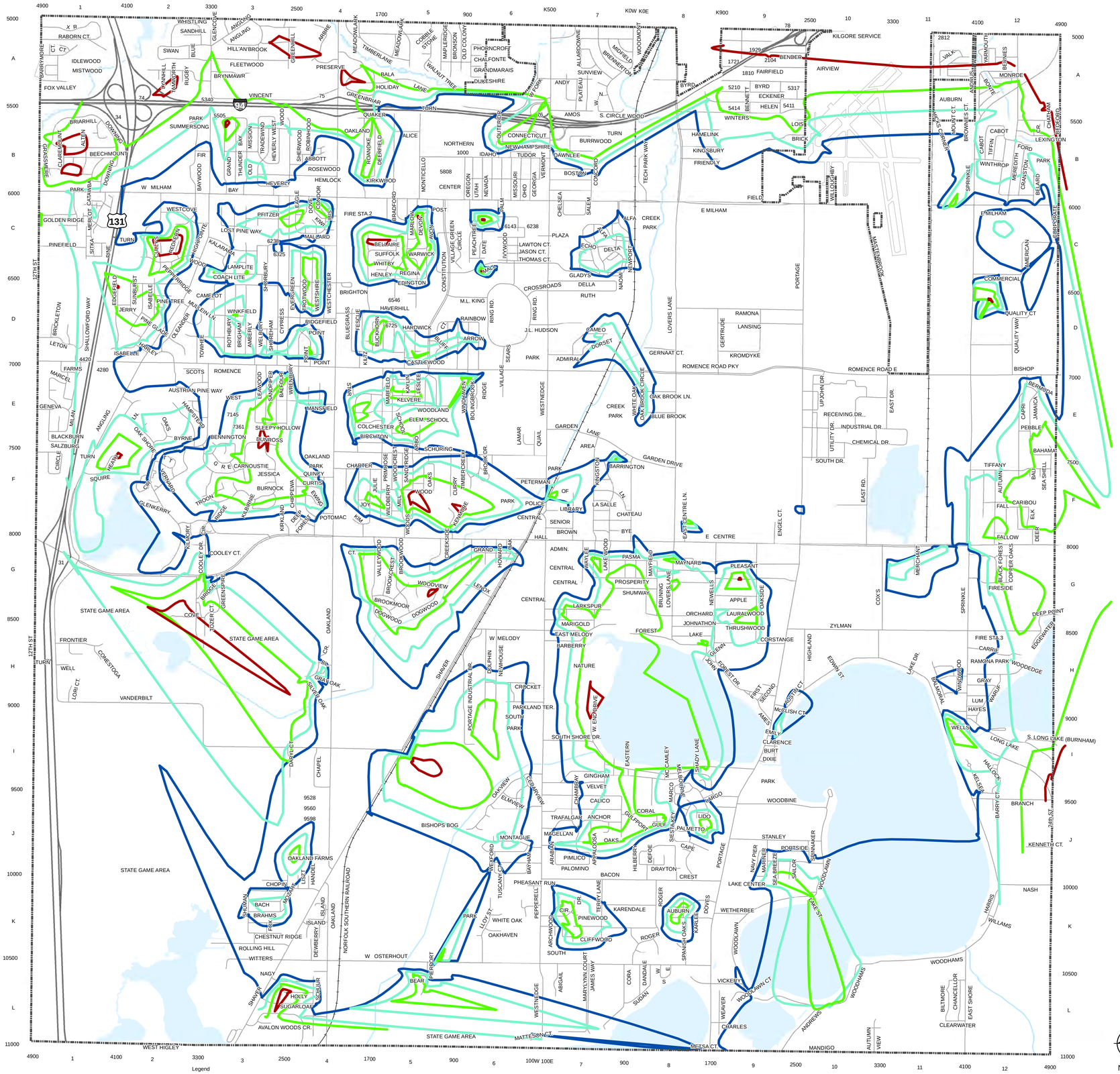
PLOT INFO: J:\2013\120751\CAD\GIS\Map Documents\S_2033PHP_PressureContour.mxd Date: 4/28/2013 11:55:56 AM User: rbg



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PROJECT NO.

G120751

FIGURE NO.

13

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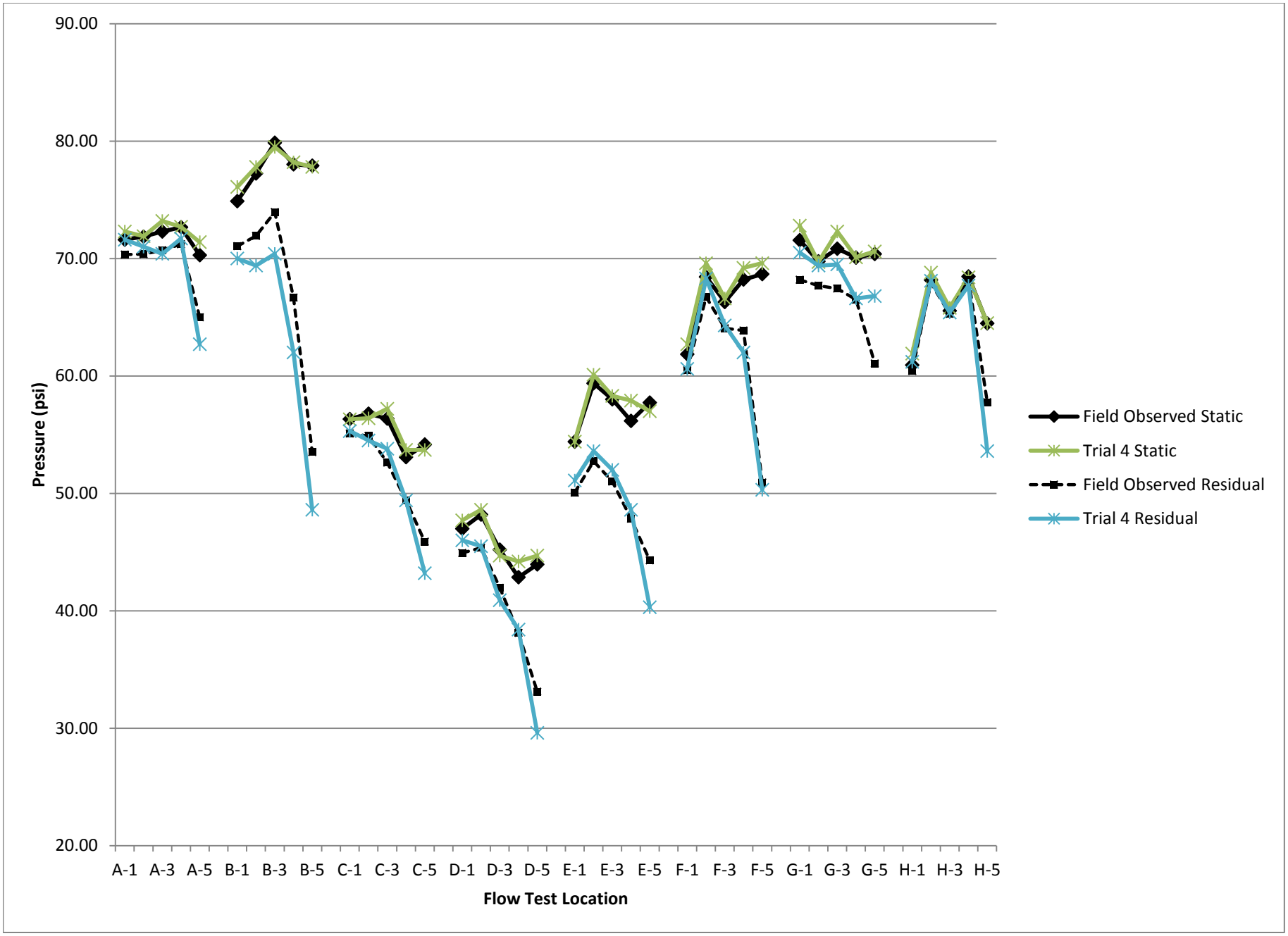
Appendix 1


90.00
80.00
70.00
60.00
50.00
40.00
30.00
20.00
Pressure (psi)
A-1
A-3
A-5
B-1
B-3
B-5
C-1
C-3
C-5
D-1
D-3
D-5
E-1
E-3
E-5
F-1
F-3
F-5
G-1
G-3
G-5
H-1
H-3
H-5
Flow Test Location
Field Observed Static
Trial 4 Static
Field Observed Residual
Trial 4 Residual