CITY OF PORTAGE
WATER SYSTEM RELIABILITY STUDY

FINAL DRAFT

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1.0 EXECUTIVE SUMMARY

The City of Portage 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 5.90 million gallons per day (mgd) in 2005 to 7.62 mgd in 2015. Maximum day demands are projected to increase from 18.83 mgd in 2005 to 24.34 mgd in 2015.

The firm capacity of the water supply wells is currently 21.51 mgd, which can adequately supply projected 2005 demands. However, three wells in the system with a combined capacity of 3.16 mgd have arsenic levels that are high enough to prohibit their use when new arsenic regulations take effect in 2006. This will reduce the firm supply capacity to 18.35 mgd. Maximum day demands are expected to be as high as 19.37 mgd in 2006, so additional water supply is immediately needed. In addition, there are two wells with a combined capacity of 2.16 mgd. Projections show that approximately 6.0 mgd of new well capacity will be required over the next ten years.

In addition to concerns over arsenic levels, water quality was also considered from an aesthetic standpoint. Iron and manganese are of particular concern in Portage. Most of the wells in the City exceed the secondary drinking water standards for iron and manganese. The secondary standards are not regulatory requirements, but are guidelines for providing water that is generally acceptable to customers. The City has already spent considerable effort searching for new water sources with acceptable iron and manganese levels, but none have yet been found. The City has also considered implementing water treatment for iron and manganese removal. There are many treatment processes that removal arsenic as well as iron and manganese that should be considered.

A hydraulic model of the water system was developed 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 Portage 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 small 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 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. One new 1.5 million gallon (MG) tank in the vicinity of the airport and one new 2.0 or 2.5 MG tank at Haverhill Park are recommended.

Development of a detailed cost estimate for new well capacity was not part of the scope of this study, but based on rough estimates, 6.0 mgd of well capacity will cost approximately $3,000,000. The total cost to build one 1.5 MG tank and one 2.0 or 2.5 MG tank is estimated at $5,450,000 to $5,950,000. The total cost of the recommended distribution system improvements to improve fire flows to 1,000 gpm or greater across the entire system, including the airport transmission main, is estimated at $1,197,000 to $1,799,000. 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 $2,340,000 to $2,720,000.

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 comparing firm water supply capacity with present and projected future water demands. This report fulfills the requirements of the State of Michigan.

In addition to the basic state requirements for a reliability study, a calibrated hydraulic model of the water distribution system was desired. A new hydraulic model of the water distribution system was developed, 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.

3.0 HISTORICAL WATER USE

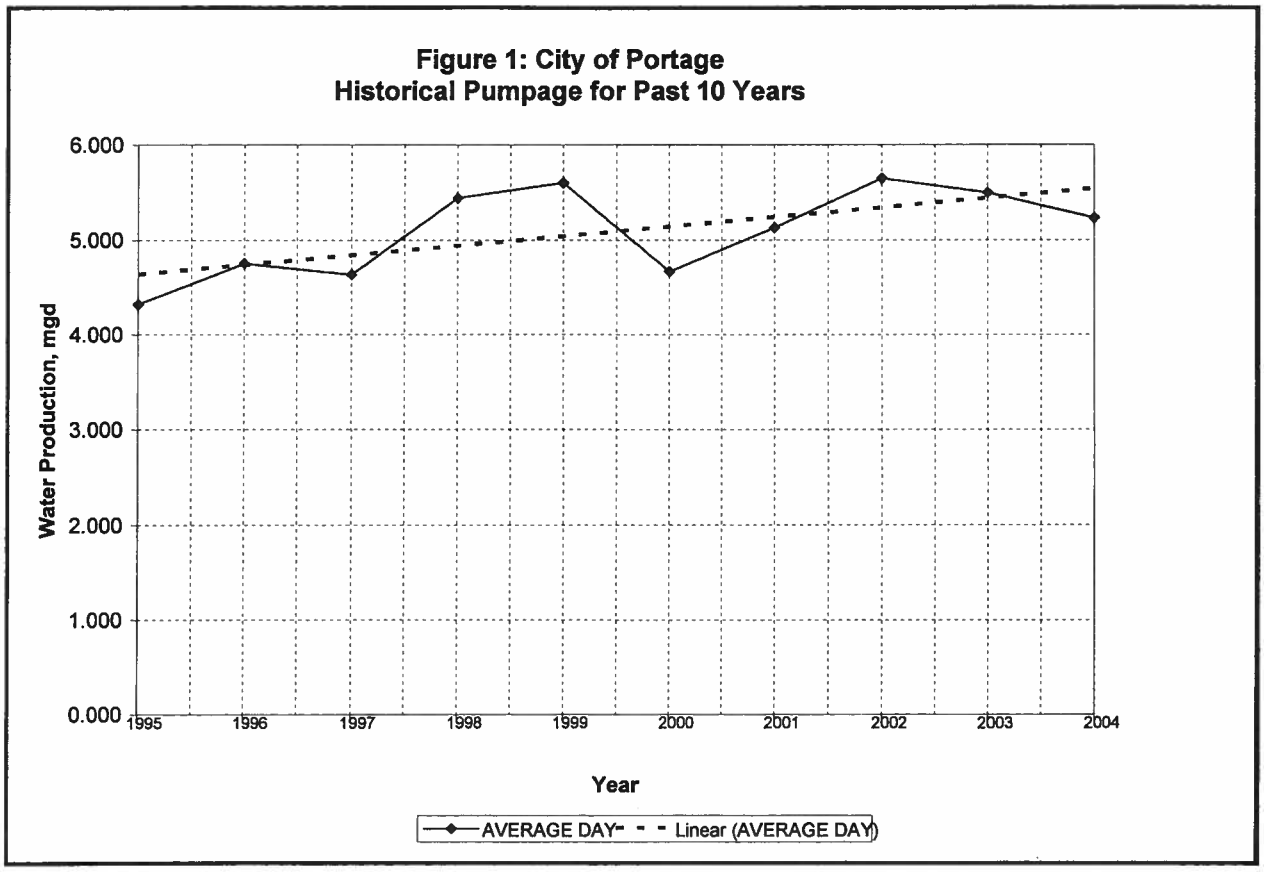
3.1 AVERAGE ANNUAL AND PER CAPITA PUMPAGE

Historical water pumpage data was compiled and reviewed for the past ten years, 1995 through 2004. 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 1990 and 2000 U.S. Census Bureau Data. The percentages of population served were obtained from the April 1995 *Water Utility Infrastructure Analysis Program Report, City of Portage.* 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 1995 Through 2004

Year	Average Day Pumpage (mgd)	Total Population	Estimated Percentage of Population Served	Estimated Serviced Population	Approximate Average Per Capita Pumpage (gpcd)
1995	4.321	42,970	59.5%	25,567	169.0
1996	4.752	43,355	61.0%	26,447	179.7
1997	4.635	43,741	62.5%	27,338	169.5
1998	5.440	44,126	64.0%	28,241	192.6
1999	5.601	44,512	65.5%	29,155	192.1
2000	4.665	44,897	67.0%	30,081	155.1
2001	5.128	45,074	68.5%	30,876	166.1
2002	5.646	45,251	70.0%	31,676	178.2
2003	5.495	45,428	71.5%	32,481	169.2
2004	5.230	45,605	73.0%	33,292	157.1
Average	5.091	--	--	--	172.9
Maximum	5.646	--	--	--	192.6
Minimum	4.321	--	--	--	155.1

Figure 1 shows a graphical representation of annual average day pumpage over the period. Average day pumpage for the past ten years has exhibited a steadily increasing trend.



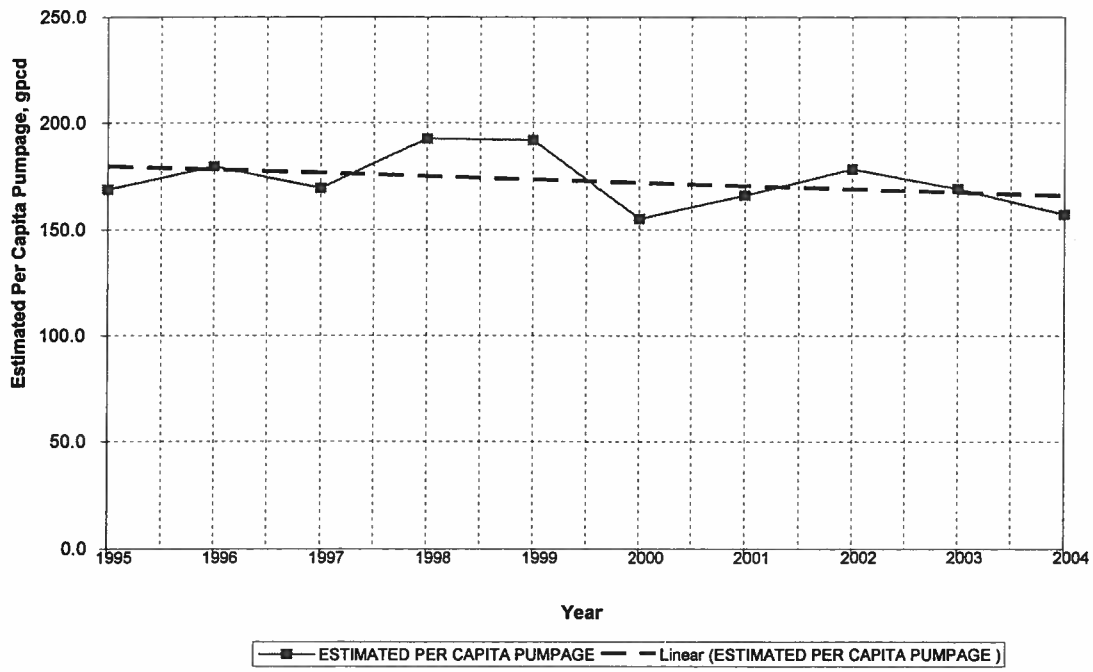
The per capita pumpage averaged 172.9 gallons per capita per day (gpcd) for the past ten years. Because any unaccounted-for water is also included in the per capita figures, they are called per capita pumpage instead of per capita usage or demand.

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 ten years.

One important figure used to calculate the per capita pumpage is the percentage of population served. These percentages were taken from the 1995 *Water Utility Infrastructure Analysis Program Report.* These type of percentages are difficult to accurately determine, and no actual data was available. While these percentages should be considered to be reasonable estimates, they are not expected to be exact. If these estimated percentages are inaccurate, the per capita pumpage will be accordingly incorrect.

Additionally, because the per 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: City of Portage
Per Capita Pumpage Trend, Past 10 Years



3.2 LARGEST WATER USERS

Table 2 shows the largest water users in the water system for 2003. The total volume of water used by the largest ten users equates to approximately 6.3% of the total volume of water pumped.

Table 2 – Summary of Largest Water Users

No.	Customer	Year 2003 Average Water Usage (gpd)	Percentage of 2003 Pumpage
1	Edward Rose	69,236	1.3%
2	Pfizer	41,485	0.8%
3	City of Portage	38,630	0.7%
4	Crossroads Mall	37,507	0.7%
5	Stryker Corp	34,795	0.6%
6	TenderCare	29,890	0.5%
7	CP Limited Partnership	27,397	0.5%
8	Portage Public Schools	24,830	0.5%
9	Charles River	22,603	0.4%
10	Oakbrook Estates	18,997	0.3%
	Total	345,370	6.3%

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.

Table 3 summarizes the total volumes of water pumped and billed from fiscal year 1993/1994 through fiscal year 2001/2002. Fiscal year data 1998/1999 was excluded because billed totals were not available for this period. In our review, it was observed that the accuracy of some data provided might be questionable. Data provided for fiscal year 1992/1993 showed that more water was billed than pumped, so this data was excluded from the analysis. The billed totals provided for fiscal 1999/2000 and fiscal 2000/2001 are identical, and appear to be a rounded off number. The percentage of unaccounted for water in fiscal 1995/1996 is much lower than any other year. For these reasons, the unaccounted-for water percentages presented in Table 3 may not accurately reflect the exact percentage of unaccounted water.

Analysis of unaccounted-for water is complicated by the fact that billing cycles and cycles for recording pumpage do not have an exact correlation. However, the average percentage of unaccounted-for water over a multi-year period should be reasonably accurate. The City is currently in the process of auditing their process for recording billed water volumes, so that unaccounted-for water can be more accurately determined in the future.

Table 3 – Summary of Unaccounted-for Water

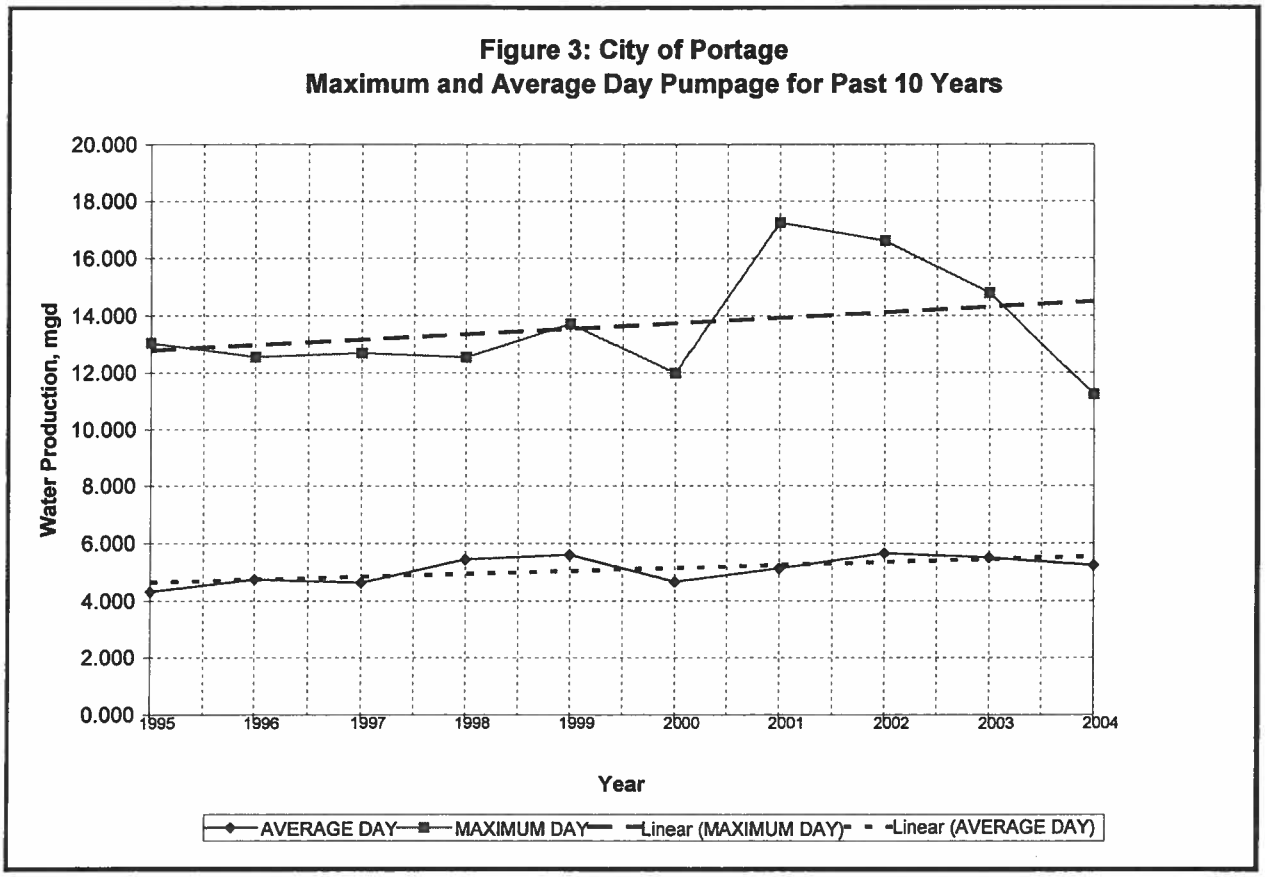
Fiscal Year	Annual Total Water	Annual Total Water Billed (mg)	Percentage Unaccounted-for
93-94	1,568.1	1,269.0	19.1%
94-95	1,468.2	1,269.0	13.6%
95-96	1,543.3	1,460.0	5.4%
96-97	1,787.0	1,574.0	11.9%
97-98	1,875.3	1,555.0	17.1%
99-00	1,886.1	1,500.0	20.5%
00-01	1,734.0	1,500.0	13.5%
01-02	1,963.0	1,540.0	21.6%
Total	**13,825.0**	**11,667.0**	**15.6%**

In our experience, percentages of lost water in the 10 to 20% range are typical. At the calculated average of 15.6%, Portage falls near the middle of this range. Many systems strive to reduce their percentage of lost 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.



The trend line in Figure 3 illustrates that maximum day demands have trended upward over the past ten years. Despite this trend, the maximum day demand for 2004 was the lowest for the past ten years. It is expected this stems from a reduction in sprinkling demands in 2004 due to cooler, wetter summer weather. Table 4 presents the average day and maximum day pumpage data used to generate Figure 3.

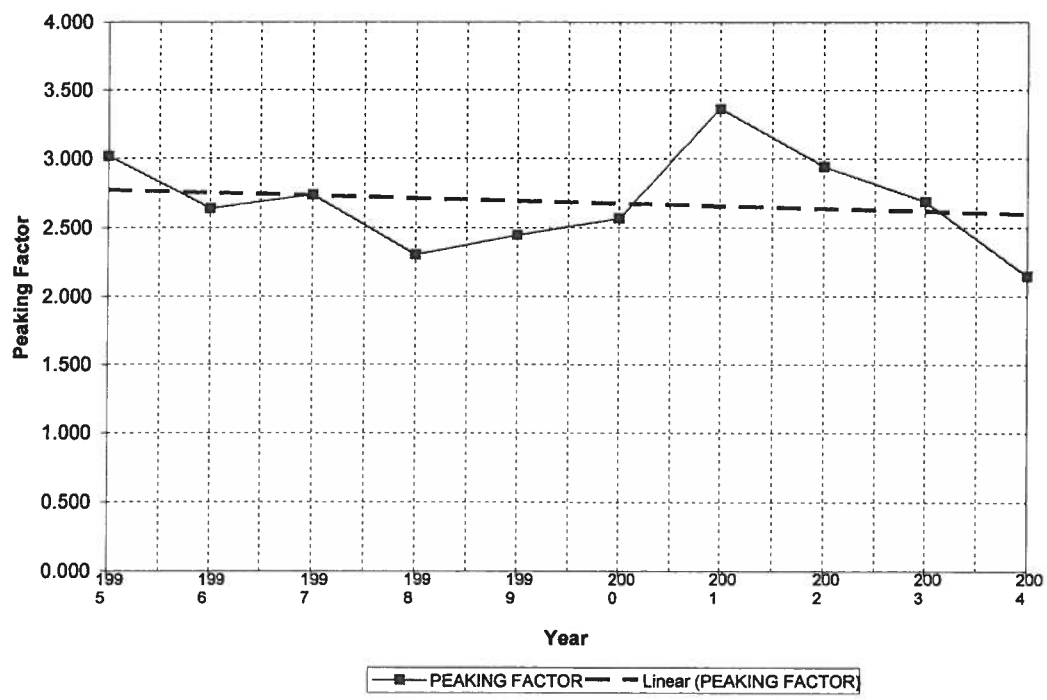
Table 4 - Maximum Day to Average Day Peaking Factors

Year	Average Day Pumpage (mgd)	Maximum Day Pumpage (mgd)	Calculated Peaking Factor
1995	4.321	13.040	3.018
1996	4.752	12.550	2.641
1997	4.635	12.690	2.738
1998	5.440	12.540	2.305
1999	5.601	13.710	2.448
2000	4.665	11.980	2.568
2001	5.128	17.240	3.362
2002	5.646	16.610	2.942
2003	5.495	14.780	2.690
2004	5.230	11.220	2.145
Average	5.091	13.64	2.686
Maximum	5.646	17.24	3.362

Minimum	4.321	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 Portage water system ranged from 2.145 to 3.362 and averaged 2.686 between 1995 and 2004. Figure 4 presents a graphical representation of the trend in peaking factors over the period. The overall trend in peaking factors over the past ten years has been slightly downward. While the average day pumpage rates have followed a relatively steady trend over the past ten years, maximum day demands have varied more erratically. This has resulted in a relatively wide variation in peaking factors.

Figure 4: City of Portage
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 Portage 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 of Portage had used peaking factors ranging from 1.50 to 1.53.

4.0 WATER DEMAND PROJECTIONS

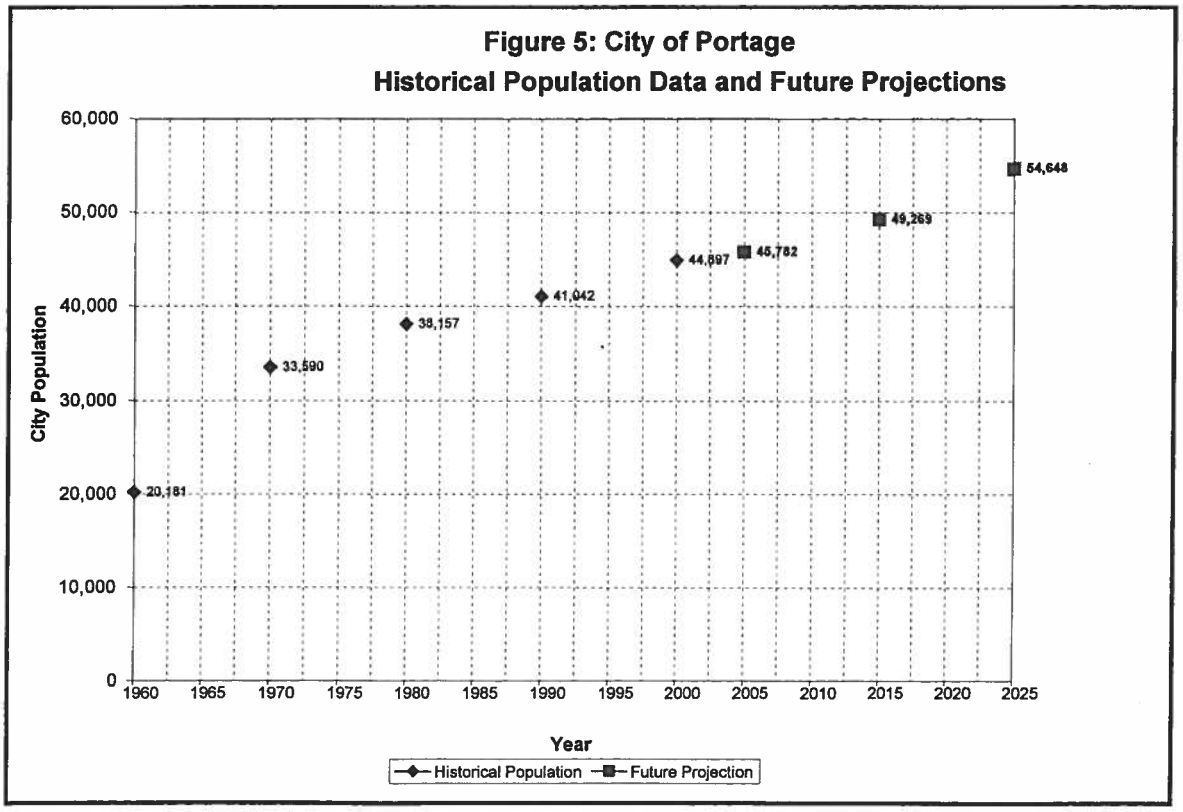
Water demand projections are typically based on changes in service population. Historical population data and future population projections were presented in the *City of Portage 2002 Comprehensive Plan* (Langworthy, Strader, LeBalanc). Table 5 summarizes historical population data and projections. The values for the percentages of population served were obtained from the previous reliability study, which estimated increases of 1.5% per year. The assumption was made that the percentage of the population served would not exceed 95%.

Table 5 – 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%	--	--
2000	44,897	--	9.4%	67%	30,081
2005	--	45,782	2.0%*	74.5%	34,108
2015	--	49,269	7.6%	89.5%	44,096
2025	--	54,648	10.9%	95.0%	51,916

*2005 percentage increase predicted for 2005 is for 5-year period vs. 10-year periods for other years.

Figure 5 provides a graphical representation of the population data.

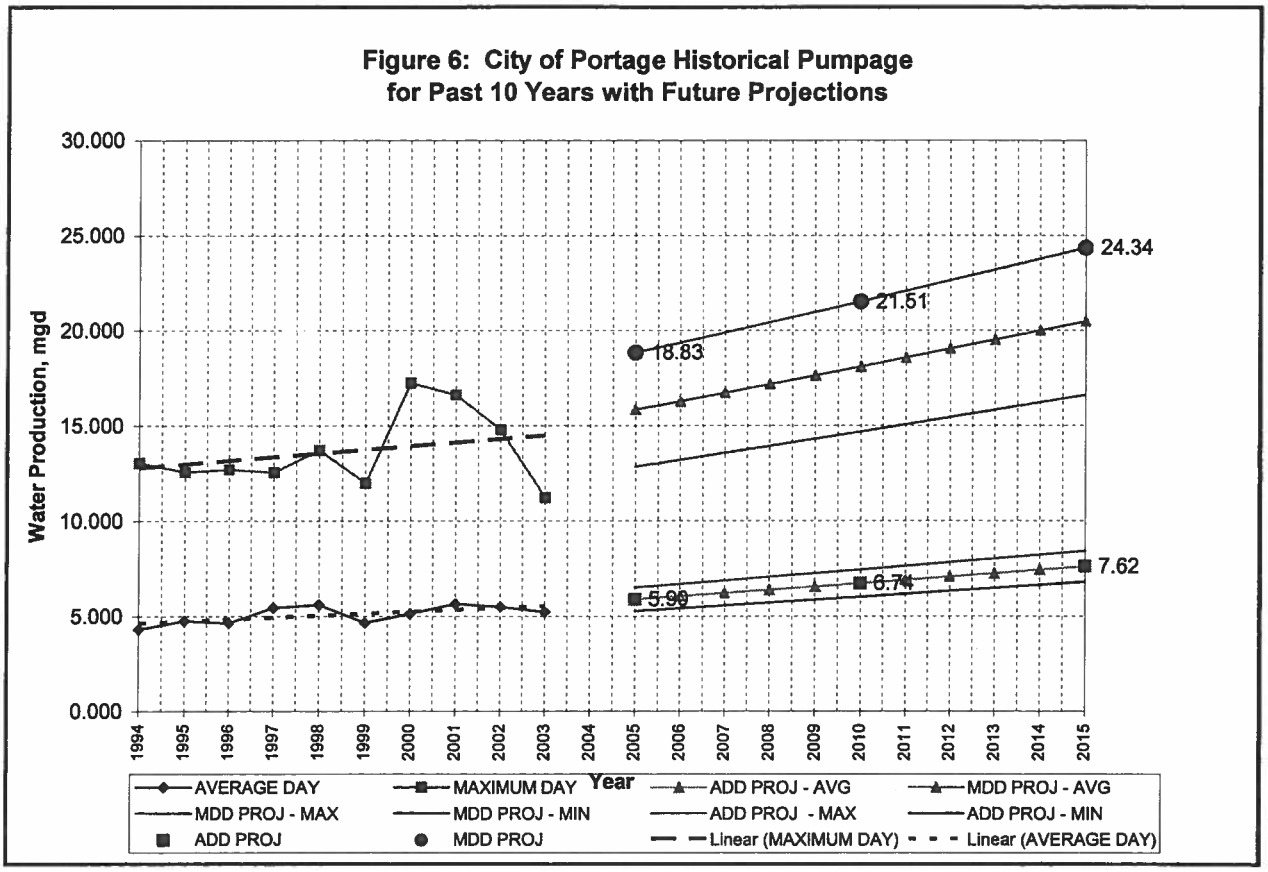


Future water use projections were developed based on an analysis of historical data and future demand projections. Because data on unaccounted-for water was not considered to be entirely accurate, the projected numbers are for pumpage, that includes both projected demand and lost water. Pumpage projections 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 form 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 6.

Table 6 – Standard Deviation Analysis

Year	Calculated Peaking Factor	Average Per Capita Per Day Pumpage
1995	3.018	169.0
1996	2.641	179.7
1997	2.738	169.5
1998	2.305	192.6
1999	2.448	192.1
2000	2.568	155.1
2001	3.362	166.1
2002	2.942	178.2
2003	2.690	169.2
2004	2.145	157.1
Average	2.686	172.9
Maximum	3.362	192.6
Minimum	2.145	155.1
Std Dev, σ	0.338	12.2
AVG + 1.5σ	3.193	191.2
AVG – 1.5σ	2.178	154.6

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 pumpage projections for 2005 and 2015. 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 2010, the maximum day pumpage is expected to be somewhere between 14.67 and 21.51 mgd.



For sizing of water system facilities, it is recommended that the upper limit of the range be used as the demand projection. Based on this convention, the projected demands for 2005 to 2015 are presented in Table 7.

Table 7 - 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
2005	5.90	18.83	3.19	28.25	1.50
2010	6.74	21.51	3.19	32.27	1.50
2015	7.62	24.34	3.19	36.51	1.50

5.0 WATER SUPPLY

5.1 SUPPLY CAPACITY

The City of Portage water supply consists of 21 supply wells, dispersed across the distribution system. Table 7 summarizes data obtained from the City on the supply of each well. Based on this data, it appears that some wells are limited by screen capacity and others are limited by pump capacity.

Although the total capacity of all wells in the system is currently capable of operating to supply system demands, new arsenic regulations will make some wells with elevated arsenic concentrations unusable in the future. The most recent arsenic levels, from 2001, are provided in Table 8. Appendix 1 provides additional data on arsenic, iron, and manganese concentrations in City wells. The City plans to discontinue use of the Lexington Green Wells and Garden Lane Well No. 2 to address concerns with arsenic. Table 8 shows the affect on system capacity if the wells with elevated arsenic levels are taken out of service.

Table 8 – Well Capacity Summary

No.	Well Name	Arsenic Level 2001 Data (µg/L)	Well Screen Capacity (mgd)	Well Pump Capacity (mgd)	Minimum of Screen and Pump Capacities (mgd)	Capacity with Arsenic Wells Removed (mgd)
1	Amberly 1	<5	0.86	1.44	0.86	0.86
2	Amberly 2	<5	1.35	0.58	0.58	0.58
3	Garden Lane 1	8	1.36	1.22	1.22	1.22
4	Garden Lane 2	29.1	1.94	1.73	1.73	0.00
5	Garden Lane 3	8	1.30	1.30	1.30	1.30
6	Garden Lane 5	11	1.30	0.94	0.94	0.94
7	Lexington Green 1	35.1	1.28	0.86	0.86	0.00
8	Lexington Green 2	25.7	1.15	0.58	0.58	0.00
9	Milham 1	4.3	1.37	1.22	1.22	1.22
10	Milham 2		2.07	1.44	1.44	1.44
11	Milham 3	6	2.72	1.44	1.44	1.44
12	Pershing 1	5	1.08	0.86	0.86	0.86
13	Pershing 2	ND	0.81	0.86	0.81	0.81
14	Portage Creek	5.8	2.45	1.73	1.73	1.73
15	Rolling Hills	3.2	1.40	1.44	1.40	1.40
16	Shuman 1		1.58	1.44	1.44	1.44
17	Shuman 2	3.6	7.20	5.76	5.76	5.76
18	Wedgewood	ND	0.86	0.58	0.58	0.58
19	Westfield 1	4.4	1.14	0.94	0.94	0.94
20	Westfield 2		0.84	0.79	0.79	0.79
21	Winterforest	8	0.81	1.15	0.81	0.81
	Total Capacity	--	34.88	28.30	27.27	24.11
	Firm Capacity	--	--	--	21.51	18.35

µg/L – micrograms per liter

In accordance with the Michigan Safe Drinking Water Act, Type 1 water systems are required to have firm supply well capacity sufficient to meet the maximum day demand. The current firm well capacity exceeds the projected 2005 maximum day demand. However, the new regulations related to arsenic will take effect in the beginning of 2006, reducing the firm capacity available for use. With the Lexington Green Wells and Garden Lane Well No. 2 taken off line, maximum day demands are projected to exceed firm capacity. Table 9 shows the minimum additional capacity required to meet projected demands. The MDEQ often recommends additional capacity be considered when demands exceed 80% of the maximum day demand, which is currently the case in Portage.

Table 9 – Supply Capacity and Demand Comparison

Year	Projected Maximum Day Demand (mgd)	Firm Supply Capacity (mgd)	Minimum Additional Capacity Required (mgd)
2005	18.83	21.51	0.0
2006	19.37	18.35	1.02
2010	21.51	18.35	3.16
2015	24.34	18.35	5.99

The data provided in Tables 8 and 9 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 be required as soon as the summer of 2006. Alternatives to increase supply capacity should be pursued now to increase capacity within the next year. The City has already completed some preliminary investigations into new well sites. While there were several sites found that may have adequate water production, preliminary data showed that iron may be present in undesirable concentrations at each of these sites.

As an alternative to increasing capacity, the City could force a reduction in maximum day demands by developing sprinkling restrictions. Although sprinkling restrictions may help alleviate the short-term supply deficit, additional capacity will still be needed to meet demands in the long-term.

One other potential area for concern is that two wells, Garden Lane Wells No. 1 and 5, have had test results that were either very close to or above the arsenic limit at some time in the past. While the most recent sampling data shows that these wells were in compliance with the arsenic standards, a slight

increase in arsenic levels could change this. The combined capacity of these two wells is approximately 2.16 mgd.

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 potentially provide a cost effective means for increasing system firm capacity. Table 10 shows several wells in the system that may potentially have additional capacity available. 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. Two wells that were identified to have the potential to increase capacity were Portage Creek and Shuman Creek 2.

5.12 WATER TREATMENT

Water with high arsenic levels can be treated to reduce arsenic to acceptable concentrations. Treating water at the Garden Lane 2 and Lexington Green 1 and 2 wells would allow these wells to continue to be used. If treatment is implemented to remove arsenic, treatment to remove iron and manganese should also be considered.

6.0 WATER STORAGE

The City of Portage 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 (United States Geological Survey [U.S.G.S.]). 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 11 – 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
2005	18.83	2.25	11.9%	1.69
2015	24.34	2.25	9.2%	1.69

The percentages shown in Table 11 are significantly below the 25% guideline. To meet the 25% guideline for the projected 2015 demands, an additional 3.84 MG of storage would be required. The 25% guideline is not a regulatory requirement, and the Portage water system currently has additional built-in reliability due to the distribution of the existing supply wells across the system.

Because storage volumes available in the system appeared to be low, a more detailed analysis was completed to determine how much additional storage would be required. This analysis included calculating the needed storage volume for equalization of diurnal flow variations over a 24-hour period, for fire flows, and for emergency purposes. Diurnal flow variations are variations in demand over a day. For example, when demands average 10 MG over the day, instantaneous demands at 8 a.m. may be 15 mgd, but demands at 2:00 a.m. might be only 5 mgd. A diurnal curve accounts for these variations. Table 12 shows the results of this analysis.

Calculating the needed storage volume for equalization of diurnal flow variations required the development of a diurnal flow curve. No data was available on current flow patterns in the City. An estimated diurnal curve was available in the 1995 report. This curve was scaled to current projected 2005, 2015, and 2025 demands and used to calculate needed volumes. A 20-year planning period was considered for elevated storage tank sizing to ensure that recommendations for the 10-year planning period of the reliability study were consistent with longer-term system needs. Two analyses of capacity were completed.

The first analysis assumed that supply capacity would match demands on a maximum day. This is consistent with the requirement that a system have firm capacity equal to maximum day demand, and assumes that the largest well in the system (Shuman Well No. 2) is out of service. The total volume required for flow equalization of diurnal fluctuations over a 24-hour period was calculated to be 2.73 MG for 2005, 3.53 MG for 2015, and 4.15 MG for 2025. The data for the first analyses are provided in Appendix 2 and summarized in Table 12.

Table 12 – Summary of Total Required Storage Volume – Analysis 1

Year	Equalization + Fire (mg)	Equalization + Fire + 4-hour Emergency (mg)	Equalization + Fire + 8-hour Emergency (mg)	Equalization + Fire + 12-hour Emergency (mg)
2005	4.17	5.15	6.14	7.11
2015	4.97	6.23	7.51	8.78
2025	5.59	7.09	8.58	10.08

The second analysis assumed the supply capacity would exceed demands by 2,000 gpm on a maximum day. This assumes that all wells in the system are in service, but that the largest pump in the system (Shuman Well Pump No. 2a or 2b) is out of service. Although there are no regulatory requirements that address supply capacity specific to fire flow, this approach was deemed the most reasonable for sizing storage facilities. The total volume required for flow equalization was calculated to be 1.47 MG for 2005, 1.95 MG for 2015, and 2.43 MG for 2025. The data for the second analysis are provided in Appendix 2 and summarized in Table 13.

The fire flow requirements were calculated based on the maximum desired fire flow of 6,000 gpm for four hours. This fire flow required 1.44 MG of elevated storage.

The emergency storage requirements were calculated based on the number of hours of average day demand that were provided in storage. Emergency storage required between 0.98 and 2.95 MG to supply

between 4 and 12 hours at 2005 average day demand conditions, and between 1.27 and 3.81 MG to supply between 4 and 12 hours at 2015 average day demand conditions.

Table 13 – Summary of Total Required Storage Volume – Analysis 2

Year	Equalization + Fire (MG)	Equalization + Fire + 4-hour Emergency (MG)	Equalization + Fire + 8-hour Emergency (MG)	Equalization + Fire + 12-hour Emergency (MG)
2005	2.91	3.89	4.87	5.86
2015	3.39	4.66	5.93	7.20
2025	3.87	5.36	6.86	8.35

As Table 13 shows, the number of hours of emergency equalization volume that are provided directly impacts the volume of elevated storage required. The City should determine which duration provides an acceptable level of risk. For the purposes of this study, four hours of emergency storage will be used as the design basis. Based on this scenario, the total storage volume required is 3.89 MG for 2005 demand conditions and 5.36 MG for 2025 demand conditions. These storage volumes equate to 21% of 2005 maximum day demands and 22% of 2025 maximum day demands.

It is recommended that the total storage volumes for the second analysis, shown in Table 13, be provided. This assumes the system capacity with the largest pump out of service will always exceed demand by at least 2,000 gpm.

Recommendations for the locations and sizing of proposed storage tanks were evaluated using the hydraulic model and are discussed in Section 8.

6.2 ELEVATED STORAGE TANK INSPECTIONS

Both elevated storage tanks were recently inspected by Dixon Engineering. The Pineview tank was inspected in 1999, was found to be in very good condition, and very minimal repairs were recommended. The Haverhill Tank was inspected in 2002 and extensive repairs were recommended. The largest repair items were recoating the interior and exterior of the tank; the estimated cost to complete all repairs to the Haverhill Tank was $460,000.

The cost of required improvements to the Haverhill Tank will be considered in the recommendations for future system improvements and compared with the cost of added elevated storage.

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 the water supplied also needs to be considered. Arsenic levels were discussed in Section 5.1. The wells with high arsenic levels have been excluded from the supply capacity, as they will soon be unusable without treatment. Water with high arsenic can be treated to reduce arsenic to acceptable concentrations. Treating water at the Garden Lane 2 and Lexington Green 1 and 2 wells would allow these wells to continue to be used. If treatment is implemented to remove arsenic, treatment to remove iron and manganese should also be considered.

Iron and manganese are aesthetic concerns for the water quality in Portage. Most of the wells in the system have iron and manganese levels that exceed the secondary drinking water standard of 0.3 milligrams per liter (mg/L) for iron and 0.05 mg/L for manganese. Table 14 summarizes some available data on iron and manganese concentrations obtained from the City.

Table 14 – Supply Well Iron and Manganese Data

Supply Well	Unit	Iron				Mn
		2002 City Data	2003 City Data	2004 City Data	1995 SEG report	1995 SEG report
Amberly # 1	mg/L	ND	ND	ND	0.00	0.04
Amberly # 2	mg/L	-	-	-	0.02	0.03
Garden Lane # 1	mg/L	1.5	0.9	1.1	1.20	0.16
Garden Lane # 2	mg/L	0.9	1.0	0.8	1.20	0.07
Garden Lane # 3	mg/L	0.9	1.3	1.7	1.80	0.43
Garden Lane # 5	mg/L	0.5	0.4	0.4	0.39	0.31
Lexington # 1	mg/L	2.4	2.0	2.4	2.50	0.12
Lexington # 2	mg/L	-	-	-	4.90	0.12
Milham # 1	mg/L	0.4	0.2	0.3	0.33	0.20
Milham # 2	mg/L	-	-	-	0.21	0.19
Milham # 3	mg/L	0.4	0.3	0.3	0.32	0.16
Pershing # 1	mg/L	0.3	0.2	0.5	1.80	0.19
Pershing # 2	mg/L	-	-	-	6.00	0.39
Portage Creek	mg/L	0.7	0.5	0.6	0.86	0.23
Rolling Hill	mg/L	0.8	0.7	0.1	2.40	0.12
Shuman # 1	mg/L	1.2	1.0	1	1.40	0.09
Shuman # 2	mg/L	2.6	2.6	-	1.30	0.11
Wedgewood	mg/L	ND	ND	ND	0.53	0.11
Westfield # 1	mg/L	0.6	0.3	0.6	0.66	0.17
Westfield # 2	mg/L	-	-	-	0.66	0.17
Winterforest	mg/L	0.5	0.5	0.4	0.72	0.14

A previous study was completed to evaluate the feasibility and cost of treatment to remove iron and manganese at the Shuman Well. Adding treatment at this well could adequately supply baseload demand in the system, but at significant cost.

An additional study was completed to determine if new supply wells with low iron concentrations could be found. For the sites investigated, preliminary data showed that significant additional supply was available, but iron levels were expected to exceed aesthetic standards.

While this report does not examine aesthetic water quality in detail, it is a consideration in the development of recommendations for system improvements.

8.0 WATER DISTRIBUTION SYSTEM

A detailed evaluation of the City of Portage water distribution system was completed as part of the Water System Reliability Study. The evaluation included development of a calibrated water system hydraulic model. The 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 GENERAL PLAN

The City of Portage has a general plan showing all water system facilities in AutoCAD® format. The plan is up to date and well-maintained.

8.2 HYDRAULIC MODEL DEVELOPMENT

A hydraulic model of the water distribution system was developed to identify any deficiencies in the current system and to assist in planning for future development of the system. Haestad Methods WaterCAD® Version 7.0 was the software used to create the model of the distribution system.

The City had an existing skeletonized hydraulic model that was developed as part of a prior study. This model contained only the largest pipes in the system, and excluded many recent improvements to the system. It addition, the model did not correlate spatially with the existing system general plan and other system base maps. Using the existing model would have made presentation of model output difficult, and would not have provided information on many areas of the system that were not included in the model. Instead, the City elected to have a new model built that was more complete and spatially accurate. The new hydraulic model was developed using data from the City's geographical information system (GIS) and water distribution system map. In total, the new model includes more than 1,600 pipes and 1,100 junctions, as compared with 350 pipes and 240 junctions in the old model.

In developing the hydraulic model, the goal was to limit the total number of pipes in the system to keep it below 2,000 pipes. This would reduce the cost of the software license that would be required. After importing pipe data from the City's GIS system, there were approximately 8,000 pipes and 7,000 junctions in the model. Pipe segments in series were merged and short, dead-end pipes were eliminated from the model to reduce the total number of pipes to approximately 1,600. This will allow the use of a 2,000-pipe software license, with an allowance for adding new pipes to the model to reflect improvements

as the system expands. While short lengths of dead-end pipe were removed from the model to reduce the total number of pipes, longer runs of dead-end pipe were retained.

All active well pumps in the system were modeled. The wells were located based on the general plan. Well pumping levels and well pump curves were entered based on the most recent data available. The elevated storage tanks were also modeled.

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 2003, 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

A fire flow analysis was completed on the uncalibrated model to identify locations where the system could be significantly stressed. In choosing the hydrant flow test locations, 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, 14 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, with the exception of 5 of 80 locations, which had discrepancies of 3 or 4 psi. Modeled residual pressures were within 4 psi of field data or better, with the exception of 2 of 80 locations, which had discrepancies of 5 psi or greater.

During the calibration process, a closed valve was identified at flow test M. When the valve was opened, the model calibrated well.

While field investigations did not reveal any closed valves, it is suggested that this area be investigated in greater detail at a later date. Flow test is in the vicinity of Centre Avenue and Parkside Road.

Appendix 4 contains a spreadsheet and graphs that summarize the calibration data. A graphic of the data has been provided for the final best fit calibration run. Calibration data also points to a potential closed valve near Location L. With the exception of Location L, the accuracy of model calibration is considered to be very good. The model can be used with confidence for analyzing the system.

To be conservative and account for areas of the system that were not flow tested, values of 90% of the calibrated C-factors were used in all subsequent model runs.

8.4 2005 DEMAND MODEL RUNS

The model was run at the projected 2005 demand conditions (18.83 mgd) to evaluate the performance of the system at current conditions. Because 2015 demands resulted in lower system pressures and available fire flows than 2005 demands, the figures presented are primarily for 2015 conditions only. The minimum desired pressure in the system is 35 psi.

8.4.1 2005 MAXIMUM DAY DEMAND MODEL RUNS

The calibrated model was run with 2005 maximum day demand conditions to evaluate the current distribution system. During the model runs, a combination of well pumps was selected to supply and keep elevated storage tank levels balanced. System pressures ranged from 41 to 82 psi during 2005 maximum day demand conditions runs. Figure 7 shows system pressures at 2005 maximum day demand conditions.

8.4.2 2005 PEAK HOUR DEMAND FLOW MODEL RUNS

The calibrated model was run with 2005 peak hour demands (28.25 mgd). During these model runs all well pumps in the system were run at full capacity, except for the Lexington Green Wells and Garden Lane No. 2. Under these conditions, both of the elevated storage tanks were draining at a high rate. After one hour, the Haverhill Tank had dropped to a level of approximately 65% full and was draining at a rate of 2,000 gpm. The Pineview Tank had dropped to a level of approximately 82% full and was draining at a

rate of 1,400 gpm. System pressures after a one hour duration were between 38 and 79 psi. System pressures were above the recommended minimum pressure of 35 psi after one hour.

8.4.3 2005 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 2005 demands (18.83 mgd).

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 exact required fire flow depends on the type of use in the area (i.e., residential, industrial, commercial, etc.). For this study, two flow rates were considered and evaluated as minimum available fire flow for all areas. 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. Fire department staff were consulted about fire flow requirements and said the available fire flows as calculated by the model for the existing system were adequate. Appendix 5 contains a memorandum from the Fire Department regarding fire flows.

8.5 2015 DEMAND MODEL RUNS

The ability of the water distribution system to serve projected increases in demands was evaluated using the model. The City identified some areas where growth was anticipated to occur. No details on proposed developments were available, but some preliminary estimates were developed. Demands at existing junctions were also increased to reflect system growth.

8.5.1 2015 Maximum Day Demand Model Runs

The calibrated model was run at 2015 maximum day demand conditions (24.34 mgd) to evaluate the capability of the existing system to meet projected demands. The analysis completed in Section 5.0 had shown that additional well supply would be required in the future to meet firm capacity requirements. The 2015 model runs assumed that this new capacity would be in place to meet demands. This was modeled by running all pumps in the system except for Lexington Green, Garden Lane No. 2, and one of the two pumps at Shuman Well No. 2. This exceeds the firm capacity of the system, since the Shuman Well is in service at half capacity. In addition, new capacity was required. Approximately 2 mgd of capacity was

added to the southeast end of the system to provide well capacity that could exceed demand. Some preliminary study has shown that additional well capacity may be available in this area of the City.

The existing system pressures ranged from 42 to 81 psi during 2015 maximum day demand conditions. Figure 8 shows graphical results for the maximum day 2015 model run with pressure contours.

8.5.2 2015 Peak Hour Demand Flow Model Runs

The calibrated model was then run at 2015 peak hour demands (36.51 mgd). The same well capacity was maintained as in the 2015 maximum day demand model runs. Under peak hour demands, both elevated storage tanks were draining at a very high rate. After a one hour duration, the Haverhill Tank had dropped to a level of approximately 39% full and was draining at a rate of 4,100 gpm. The Pineview Tank had dropped to a level of approximately 76% full and was draining at a rate of 2,900 gpm. System pressures after a one hour duration were between 33 and 76 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 there were expected to fall below the recommended minimum pressure of 35 psi. Figure 9 shows graphical results for the 2015 peak hour model run with pressure contours after a one hour duration.

8.5.3 2015 Available Fire Flow Model Runs

An analysis of the available fire flow was also evaluated for maximum day 2015 demands (24.34 mgd). The same pumps were operated as in the 2015 maximum day demand runs. The available fire flows were similar to 2005, but slightly lower at most locations. In general, the areas with low available fire flows were enlarged slightly over 2005. 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 deficient. On the northwest end of the system, the area in the vicinity of Westfield Park has pressures that fall below 35 psi at 2015 peak hour

demands. A larger surrounding area has pressures between 35 and 40 psi, which are acceptable but marginal. This is primarily due to the high elevation of this area relative to the rest of the system.

8.6.2 Fire Flow Deficiencies

There are a few specific areas with fire flow deficiencies that are worth noting. On the north side of the City west of the Kalamazoo Municipal Airport, the area north of Winters Drive has available fire flows between 750 and 1,350 gpm. Available fire flows in this area are limited by 6-inch and 8-inch mains in this area, and by the single 8-inch main on Winters Drive that serves it.

On the northwest end of the system, in the vicinity of Cedarcrest and Briarhill Streets, available fire flows are below 1,300 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. Available fire flows in this area are limited by a single 8-inch main serving this area that is essentially a dead-end.

On the southeast end of the system, near Woodlawn Avenue and Stanley Street, there is a small area with fire flows less than 1,500 gpm. Available fire flows in this area are limited by long runs of 8-inch mains.

It is important to note that many short, dead-end segments were eliminated from the model to reduce the total number of pipes. While these dead-end pipes have fire flows that are less than desired, they have not been analyzed by the model and will not be described in this report.

8.7 PROPOSED SYSTEM IMPROVEMENTS AND MODEL RUNS

Improvements were developed to remedy deficiencies in growth areas and in the portions of the existing system with inadequate fire flows. The model was used to evaluate improvements and determine which were most effective. Hydraulic modeling for the future improvement scenarios was completed using the 2015 projected demands, and assuming additional supply capacity as discussed in Section 8.5.1.

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 new elevated storage tanks.

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. A second pipe improvement scenario evaluated improvements to increase available fire flows to greater than 1,500 gpm in all areas. The proposed airport transmission main has also been included in both improvement scenarios. The City has planned the airport transmission main for construction in the near future. The airport transmission main is discussed in greater detail in the *Northeast Water Storage Facility Preliminary Design Report* (FTC&H, 2005). 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	Airport Transmission	16	2,100
A-2	Portage Road, Milham to Byrd	12	4,200
A-3	West End - North of South Shore	8	1,000
A-4	Point-O-Woods and Point-O-Woods Ct.	8	1,000
A-5	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	Stanley Avenue - Portage to Spinnaker	12	2,500
B-3	Fawn Cove Road - West of Shirley Court	8	1,100
B-4	Brynmawr Road - East of Angling Road	10	1,200
B-5	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 3.89 MG for 2005 demand conditions and 5.36 MG for 2025 demand conditions. The hydraulic model was used to evaluate ten locations for new elevated storage tanks across the system. Flow rates out of tanks at peak hour demands were considered at each site. Some sites were superior to others due to their proximity to high demand areas and large diameter pipe.

8.7.2.1 PROPOSED ELEVATED STORAGE IMPROVEMENT LOCATIONS

The approximate locations for elevated storage that were determined to be the most viable from a system hydraulics standpoint were: Haverhill Park, Angling Road, and Milham Road, Sprinkle Road and Commercial Drive (Airport Site 4), and Centre and Lovers Lane. The Haverhill Park site was determined to be the best location.

8.7.2.2 NEW ELEVATED STORAGE TANK AT HAVERHILL PARK

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. The required size of the Haverhill Park Tank also depends on the size and location of other storage tanks within the system. Several model runs were completed to evaluate different locations and sizes. One scenario that worked quite well was a new tank in the vicinity of the airport, and a new tank at Haverhill Park. Based on recommendations provided in the *Northeast Water Storage Facility Preliminary Design Report*, a 1.5 MG tank was modeled near the airport, in addition to the 1.5 MG Pineview Tank. Total tank volumes of 1.5, 2.0, and 2.5 MG were considered at the Haverhill Park Site.

The total storage volume at Haverhill Park could be provided with either a new tank in addition to the existing 0.75 MG tank, or with a new larger tank that would replace the existing tank. The existing Haverhill Tank requires $460,000 in repairs, and is over 40 years old. It may be advisable to demolish the

existing tank and build a larger one than to repair the old tank and build a new one with a smaller volume. For example, the difference in cost between a 2.0 and a 2.5 MG tank is approximately $500,000. Scaled up for a 0.75 MG volume, the cost of adding the additional storage to a new tank is likely near $750,000. While the cost of adding additional storage to a new tank is higher than repairing the existing tank, it is still recommended that a single tank be provided at Haverhill Park. Future maintenance costs would also likely be lower on a single large tank than on multiple smaller tanks.

Modeling showed that the ideal tank volume for equalizing peak hour demands while keeping similar levels between tanks was 2.5 MG. The 1.5 and 2.0 MG tank were also feasible options, but the differential levels between tanks increased as the tank volume decreased. Rounding up values to the nearest 0.5 mg, 2015 demands would require 5.0 MG of storage and 2025 demands would require 5.5 MG of storage. With the 1.5 MG Pineview Tank and assuming a new 1.5 MG tank near the airport, the required volume for the new Haverhill Tank is 2.0 MG to meet 2015 demands or 2.5 MG to meet 2025 demands. Water quality and turnover should also be considered when sizing elevated storage; significant oversizing is not recommended.

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. 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 2015 peak hour demand conditions after one hour.

In both Figures 12 and 13, proposed 1,000 gpm piping improvements have been highlighted and numbered with the identification numbers from Table 15.

One item that is important to point out is that system pressures will continue to drop from the values shown in Figure 5 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 2015 peak hour demands.

Table 17 – Minimum System Pressures in Westfield Area

Condition	Haverhill Tank Elevation (feet)	Minimum System Pressure (psi)
Tank Full	1034	40.7
Tank 2/3 Full	1021	36.3
Tank 1/2 Full	1014	33.9
Tank almost empty	995	26.3

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 of Portage. The following summarizes these recommendations, with estimated costs where available:

9.1 INCREASE WATER SUPPLY CAPACITY

Demands are projected to exceed firm water supply capacity as soon as 2006. To meet 2015 firm capacity requirements, approximately 6.0 mgd of new well capacity will be required. Supply capacity could be increased by constructing new wells or by treating water from wells having high arsenic concentrations.

Detailed cost estimates for increasing water supply capacity were not estimated. Based on preliminary cost estimates developed in the *Northeast Water Storage Facility Preliminary Design Report*, 6.0 mgd of well capacity may be in the range of $3,000,000 if this capacity can be provided with three or fewer large wells. This cost could vary significantly based on the location chosen for new supply wells and the number of wells required. This estimate assumes that new well capacity would be located relatively close to large diameter mains in the system, and does not include costs for new transmission main. The cost to provide treatment for any new wells would significantly increase the total cost of supply. 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 as soon as possible. Supply well capacity could be increased by installing treatment to remove arsenic, adding new wells, installing larger pumps at existing wells, or a combination of these three options. 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.

9.2 INCREASE ELEVATED STORAGE CAPACITY

System elevated storage capacity should be increased to a total of 5.0 to 5.5 MG. A new 1.5 MG tank in the vicinity of the airport and a 2.0 or 2.5 MG tank at Haverhill Park is recommended. The costs presented provide allowances for contingencies and engineering. Appendix 5 provides a summary of cost estimates.

1.	1.5 MG Airport Area Tank	$2,630,000
2.	2.0 or 2.5 MG Haverhill Park Tank	$2,720,000 or $3,220,000
3.	Demolish 0.75 MG Haverhill Park Tank and Restore Site	$100,000
	Elevated Storage Total	$5,450,000 to $5,950,000

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 Figures 11 and 12. Projects have been numbered in order of approximate priority. Estimated costs are based on unit costs per lineal foot (lft.) Unit costs of $110, $125, and $135 per linear foot were used for 8-inch, 10-inch and 12-inch main respectively. Costs for the airport transmission main were estimated as part of the *Northeast Water Storage Facility Preliminary Design Report*. These unit costs include allowances for engineering and contingencies. Appendix 5 provides a summary of cost estimates.

A-1	Airport Transmission Main (2,100 lft. of 16-inch)	$540,000 to $920,000
A-2	Portage Road from Milham to Byrd (4,200 lft. of 12-inch)	$567,000
A-3	West End - North of South Shore (1,000 lft of 8-inch)	$110,000
A-4	Point-O-Woods and Point-O-Woods Ct. (1,000 lft of 8-inch)	$110,000
A-5	School Road – Loop South to Snowberry (200 lft. of 8-inch)	$22,000
	Distribution System Improvements Total	$1,349,000 to $1,729,000

For reference, the total additional cost to complete the projects needed to improve fire flows in all areas to 1,500 gpm or greater is $883,000. The total estimated range of costs to complete both the 1,000 gpm and 1,500 gpm improvements is $2,232,000 to $2,612,000.

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 3.2, 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 FURTHER INVESTIGATE SYSTEM CALIBRATION NEW FLOW TEST "L"

The hydraulic model does not match up with field data collected at this location, which is in the vicinity of Centre Avenue and Oakside Road. This area should be investigated in further detail to see if the cause of

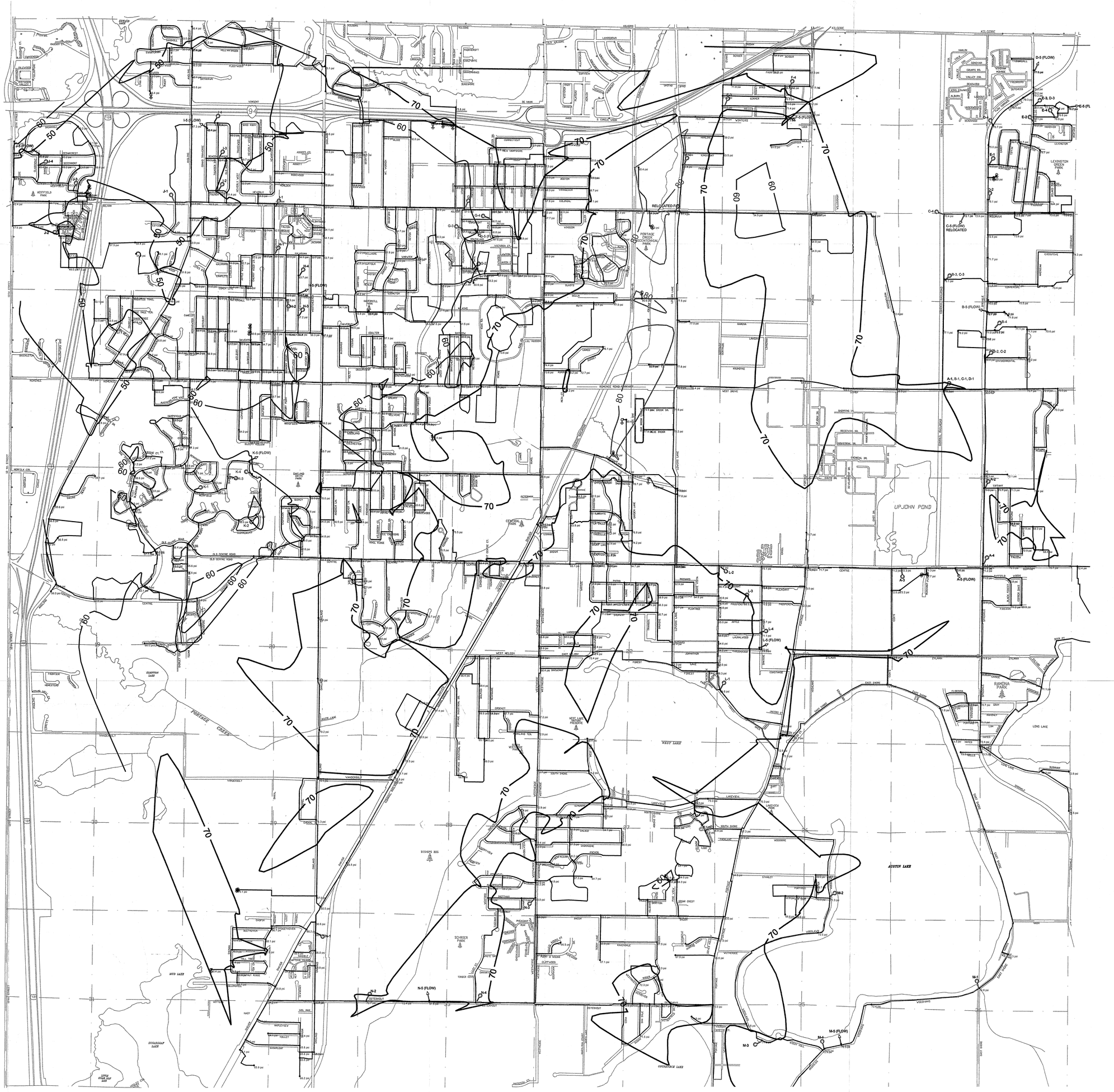
the discrepancy can be determined. Figure 7 shoes the location of the hydrants used in Flow Test L (L-1 through L-5).

9.6 PRIORITIZATION OF IMPROVEMENTS

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

1. Increase well supply capacity.
2. Construct 2.0 to 2.5 MG elevated storage tank at Haverhill Park.
3. Construct Airport transmission main.
4. Construct 1.5 MG Airport elevated storage tank.
5. Construct remaining distribution system improvement projects in the approximate order listed previously.

Work to develop new well supply capacity should begin as soon as possible, as a deficit in firm capacity is projected as soon as the summer of 2006. Projections show that an additional 1.0 mgd will be required by 2006, 3.0 mgd by 2010, and 6.0 mgd by 2015.



Existing Water System Model
2005 Maximum Day Demand (18.8 MGD)
Available Pressure Contours and Flow Test Locations

NORTH

SCALE: 1" = 1200'
0 600 1200 2400

Drafter SG
Designer DJB
Reviewer
Manager

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City of Portage
Kalamazoo County, Michigan

Water System Reliability Study

PROJECT NO.
G04071

SHEET NO.
7

PLOT INFO: N:\04071\PRECO\FIGURES - RELIABILITY STUDY BP\FIGURE7.DWG DATE: 4/7/2005 TIME: 3:54:17 PM USER: SG

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N:\04071\PRECO\FIGURE 6 BP.DWG
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N:\04071\PRECO\TCOM 5 FITCHMOOD.DWG
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N:\04071\PRECO\FIGURE 5 BP.DWG
N:\04071\PRECO\FIGURE 6 BP.DWG
N:\04071\PRECO\FIGURE 7 BP.DWG
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N:\04071\PRECO\TCOM 5 FITCHMOOD.DWG
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N:\04071\PRECO\FIGURE 5 BP.DWG
N:\04071\PRECO\FIGURE 6 BP.DWG
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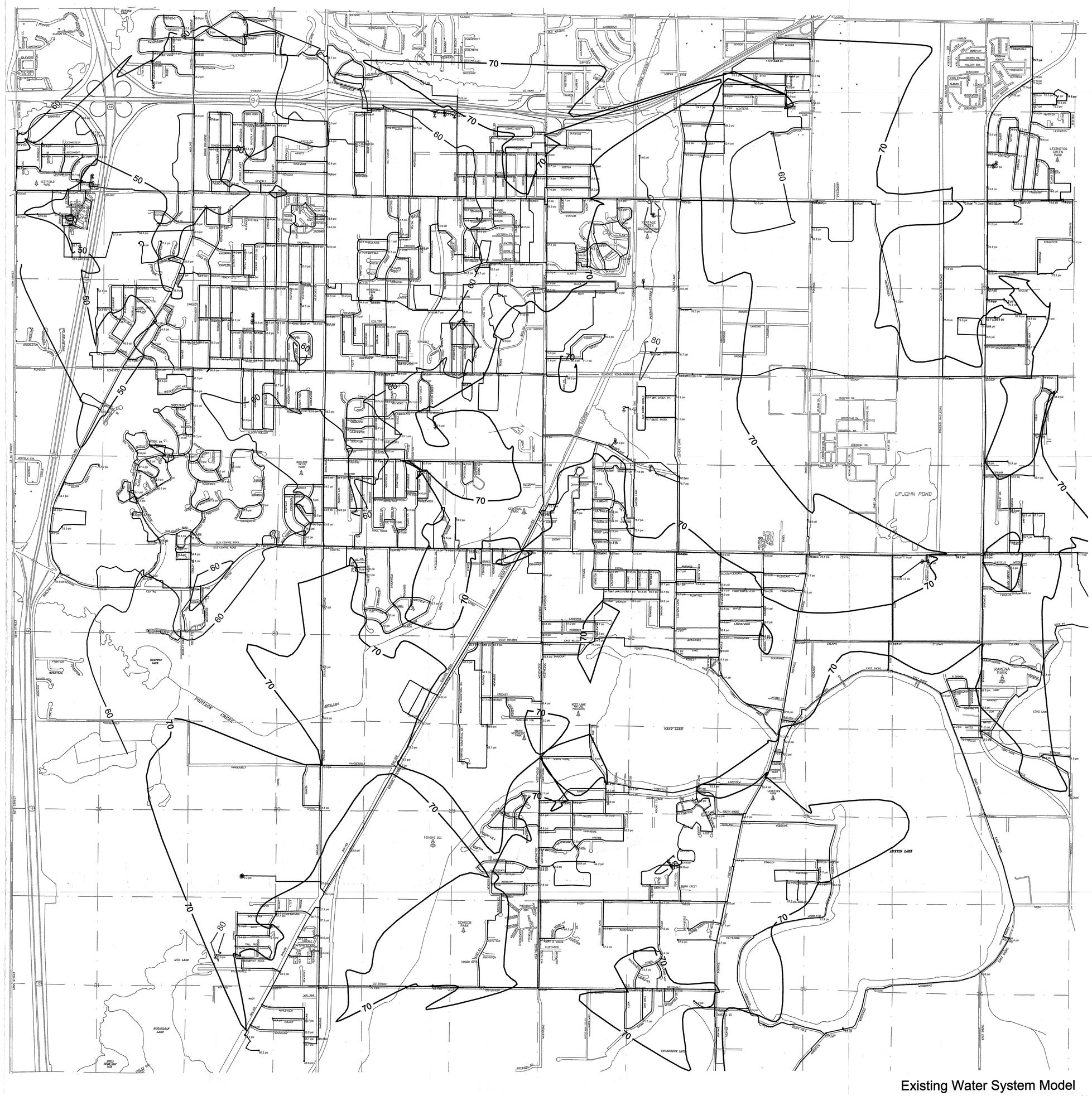

Existing Water System Model
2015 Maximum Day Demand (24.3 MGD)
Available Pressure Contours

NORTH

SCALE: 1" = 1200'
0 600 1200 2400

Drafter SG
Designer DJB
Reviewer
Manager

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City of Portage
Kalamazoo County, Michigan
Water System Reliabilty Study

PROJECT NO.
G04071
SHEET NO.
8

PLOT INFO: N:\04071\PRECO\FIGURES - RELIABILITY STUDY RPT\FIGURE8.DWG DATE: 4/7/2005 TIME: 3:43:27 PM USER: SG

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AIRPORT_AREA

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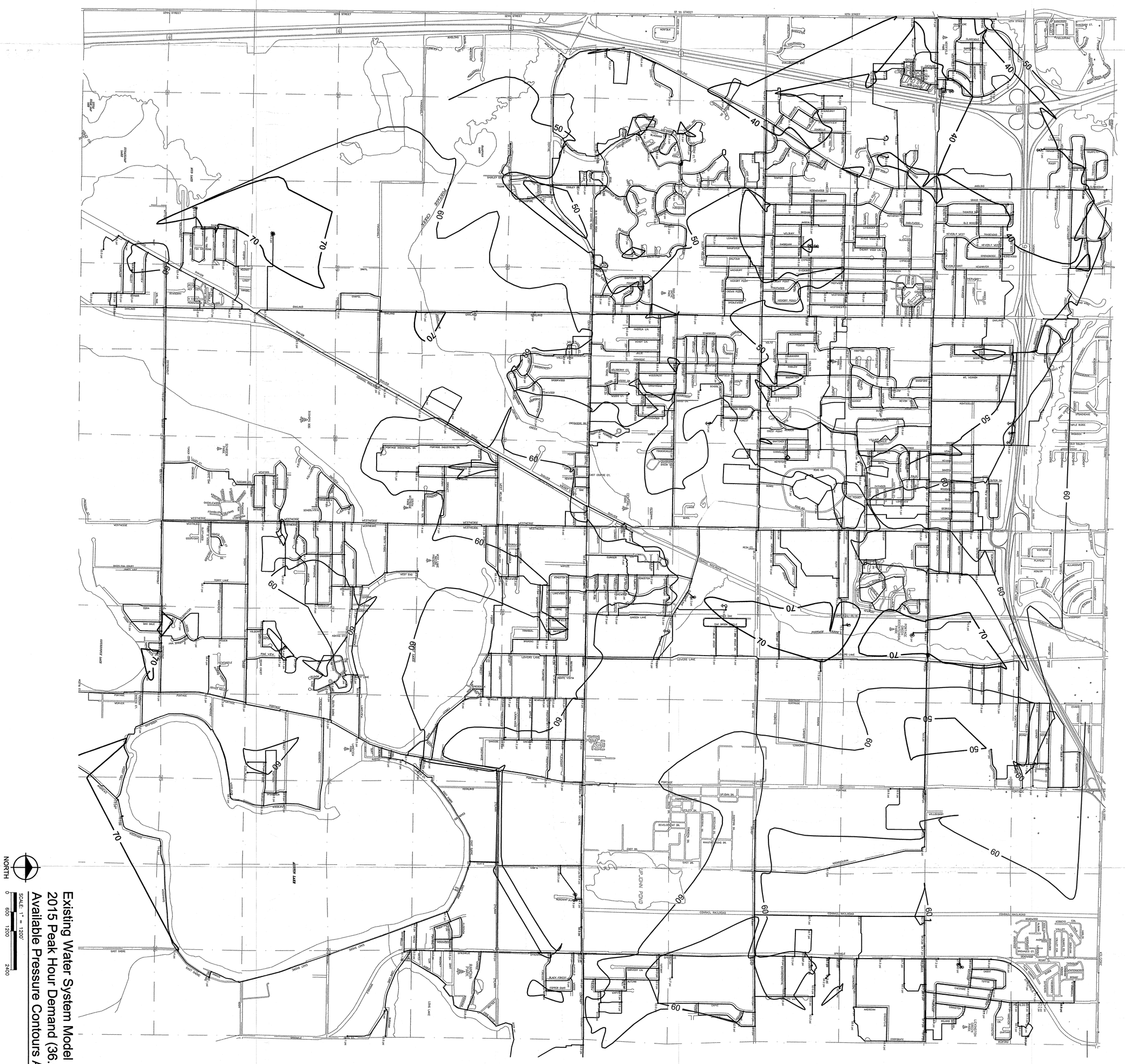

Existing Water System Model
2015 Peak Hour Demand (36.5 MGD)
Available Pressure Contours After 1.0 Hour

PROJECT NO. G04071
SHEET NO. 9

City of Portage

Kalamazoo County, Michigan

Water System Reliabilty Study

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AIRPORT_AREA

PLOT INFO: N:\04071\PRECD\FIGURES - RELIABILITY STUDY RPT\FIGURE 10.DWG DATE: 4/7/2005 TIME: 4:23:54 PM USER: SG

NORTH

SCALE: 1" = 1200'
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Existing Water System Model
2015 Maximum Day Demand (24.3 MGD)
Available Fire Flow Contours

PROJECT NO. G04071
SHEET NO. 10

City of Portage
Kalamazoo County, Michigan

Water System Reliabilty Study

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Proposed 1,000 GPM Improvements and Tanks
2015 Maximum Day Demand (24.3 MGD)
Available Fire Flow Contours

NORTH

SCALE: 1" = 1200'
0 600 1200 2400

PROJECT NO.	SHEET NO.
G04071	11

City of Portage
Kalamazoo County, Michigan
Water System Reliability Study

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Drafter	SG
Designer	DJB
Reviewer	
Manager	

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PLOT INFO: N:\0407\FIRECD\FIGURES - RELIABILITY STUDY REPT\FIGURE 11.DWG DATE: 4/7/2005 TIME: 4:28:53 PM USER: SG

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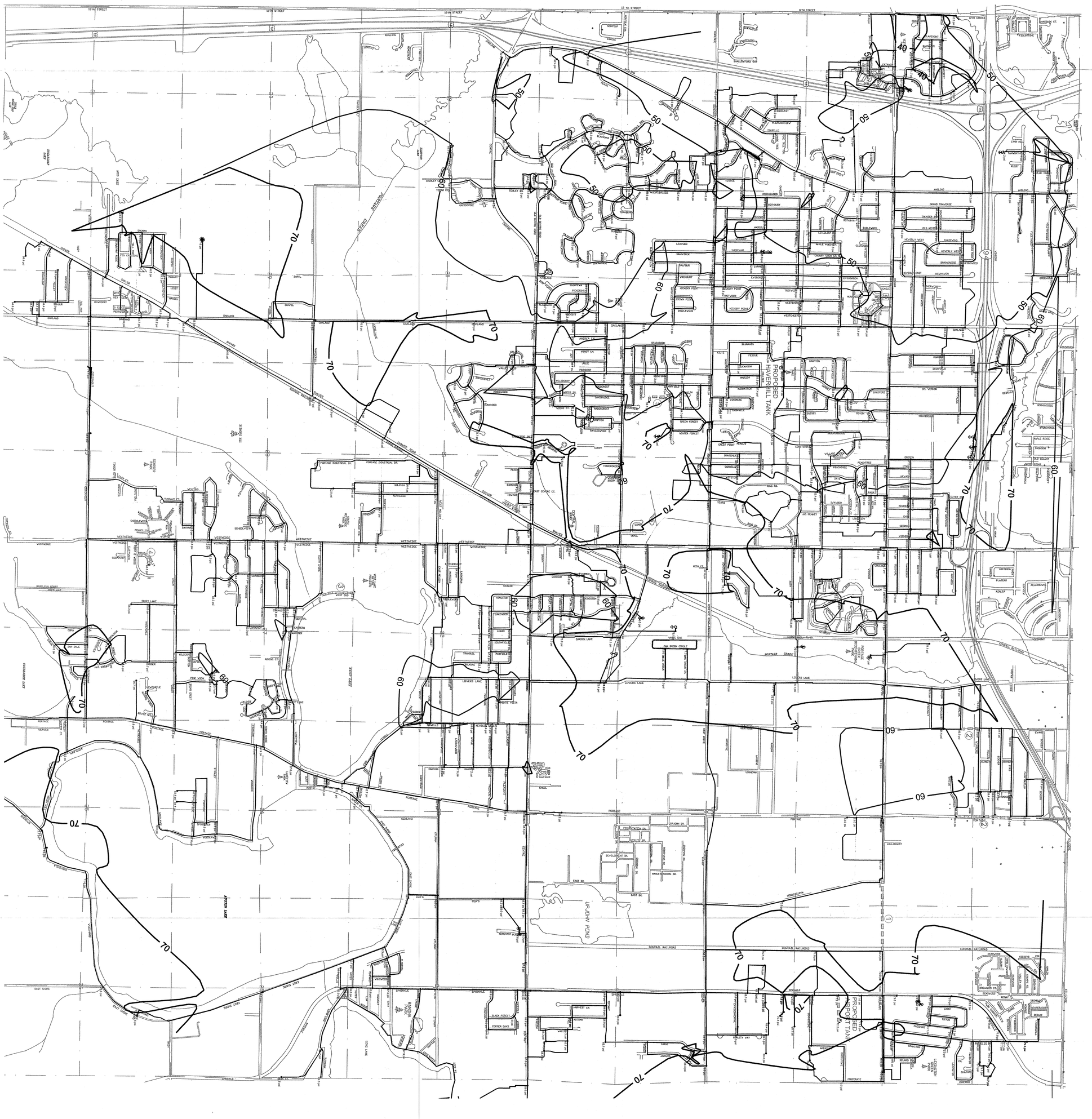
PLOT INFO: N:\04071\PRECD\FIGURES - RELIABILITY STUDY RPT\FIGURE 12.DWG DATE: 4/7/2005 TIME: 4:55:16 PM USER: SG



Improved System
2015 Peak Hour Demand (36.5 MGD)
Available Pressure Contours

NORTH

SCALE: 1" = 1200'
0 600 1200 2400

PROJECT NO. G04071	**City of Portage** Kalamazoo County, Michigan Water System Reliabilty Study	ftc&h Fishbeck, Thompson, Carr & Huber Engineers • Scientists • Architects Grand Rapids, Michigan (616) 575-3824 © Copyright 2005 All Rights Reserved	Drafter SG Designer DJB Reviewer Manager Hard copy is intended to be 30"x42" when plotted. Scale(s) indicated and graphic quality may not be accurate for any other size.
SHEET NO. 12			

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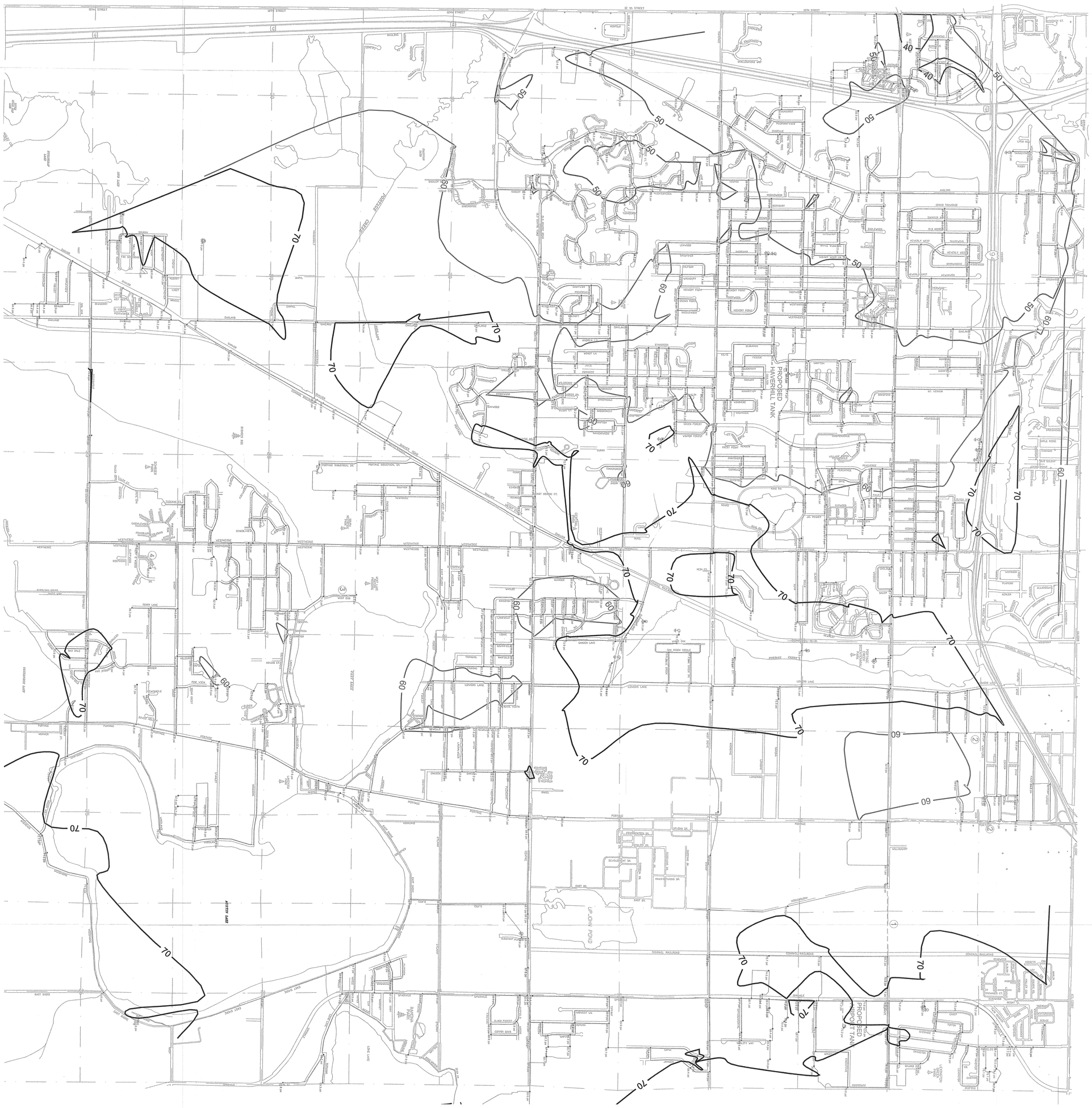
AIRPORT_AREA

PLOT INFO: N:\04071\PRECD\FIGURES - RELIABILITY STUDY RPT\FIGURE 12.DWG DATE: 4/7/2005 TIME: 4:55:16 PM USER: SG



NORTH

SCALE: 1" = 1200'
0 600 1200 2400

Improved System
2015 Peak Hour Demand (36.5 MGD)
Available Pressure Contours

PROJECT NO. G04071
SHEET NO. 13

City of Portage

Kalamazoo County, Michigan

Water System Reliabilty Study

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Appendix No. 2
City of Portage - Northeast Storage Facility Preliminary Design Report
Project No. G04071

1. Peak Hour Equalization				
Year	2005	2015	2025	
MDD	18.83	24.34	28.66	
PKHR	28.245	36.51	42.99	
Storage Volume Used - gallons	2,729,105	3,527,477	4,153,804	
2. Fire Flow				
Largest Fire Flow Demand - gpm	6000	6000	6000	
Duration - hours	4	4	4	
Volume - gallons	1,440,000	1,440,000	1,440,000	
3.a. 4 hour Emergency				
Year	2005	2015	2025	
ADD	5.9	7.62	8.97	
Duration of emergency	4	4	4	
Daily Volume - gallons	983,333	1,270,000	1,495,000	
3.b. 8 hour Emergency				
Year	2005	2015	2025	
ADD	5.9	7.62	8.97	
Duration of emergency	8	8	8	
Daily Volume - gallons	1,966,667	2,540,000	2,990,000	
3.c. 12 hour Emergency				
Year	2005	2015	2025	
ADD	5.9	7.62	8.97	
Duration of emergency	12	12	12	
Daily Volume - gallons	2,950,000	3,810,000	4,485,000	
Total				
1 + 2	4,169,105	4,967,477	5,593,804	Operational Storage + 4 hour 6000 gpm fire
1+ 2 + 3.a.	5,152,438	6,237,477	7,088,804	Operational Storage + 4 hour 6000 gpm fire + 4 hour ADD reserve (1.25 hour MDD Reserve)
1+ 2 + 3.b.	6,135,771	7,507,477	8,583,804	Operational Storage + 4 hour 6000 gpm fire + 8 hour ADD reserve (2.5 hour MDD Reserve)
1+ 2 + 3.c.	7,119,105	8,777,477	10,078,804	Operational Storage + 4 hour 6000 gpm fire + 12 hour ADD reserve (3.75 hour MDD Reserve)
Compare to 25% MDD				

Appendix No. 2:
City of Portage - Northeast Storage Facility Preliminary Design Report
Project No. G04071

Demand	
18.83	Total - mgd
13,076	Total - gpm

Hour No.	Hour	Flow from Report (gpm)	Ratio	Projected 2005 Hourly Demand (gpm)	Projected 2005 Cumulative Demand (gpm)	Assumed Supply (gpm)	Net Surplus (Deficit)	Cumulative Surplus (Deficit)
1	6	6100	0.638	8,349	8,349	8,349	0	0
2	7	10400	1.089	14,234	22,583	0	-69,459	-69,459
3	8	13500	1.413	18,477	41,060	0	-324,030	-393,489
4	9	14600	1.528	19,982	61,042	0	-414,361	-807,851
5	10	14600	1.528	19,982	81,025	0	-414,361	-1,222,212
6	11	13800	1.444	18,887	99,912	0	-348,666	-1,570,877
7	12	13800	1.444	18,887	118,800	0	-348,666	-1,919,543
8	13	12200	1.277	16,698	135,497	0	-217,274	-2,136,818
9	14	9100	0.952	12,455	147,952	0	37,296	-2,099,522
10	15	9100	0.952	12,455	160,407	0	37,296	-2,062,226
11	16	9400	0.984	12,865	173,272	0	12,660	-2,049,566
12	17	10200	1.068	13,960	187,232	0	-53,036	-2,102,601
13	18	13200	1.382	18,066	205,299	0	-299,394	-2,401,995
14	19	10300	1.078	14,097	219,396	0	-61,247	-2,463,243
15	20	12100	1.266	16,561	235,957	0	-209,063	-2,672,305
16	21	10200	1.068	13,960	249,917	0	-53,036	-2,725,341
17	22	9600	1.005	13,139	263,056	0	-3,764	-2,729,105
18	23	7700	0.806	10,539	273,595	0	152,263	-2,576,841
19	0	7700	0.806	10,539	284,133	0	152,263	-2,424,578
20	1	6800	0.712	9,307	293,440	0	226,171	-2,198,407
21	2	3400	0.356	4,653	298,094	10,136	328,954	-1,869,454
22	3	4300	0.450	5,885	303,979	5,885	0	-1,869,454
23	4	3900	0.408	5,338	309,317	5,338	0	-1,869,454
24	5	3300	0.345	4,517	313,833	4,517	0	-1,869,454
					Maximum Surplus (Deficit):			-2,729,105

Notes:
1. Determine Peak hour equalization requried based on dirurnal curve.
2. Assume Capacity equals maximum day demand.
3. Hourly ratios from prior water system study.

Appendix No. 2:
City of Portage - Northeast Storage Facility Preliminary Design Report
Project No. G04071

				24.34 Total - mgd **16,903** Total - gpm				
Hour No.	Hour	Flow from report (gpm)	Ratio	Projected 2015 hourly demand (gpm)	Projected 2015 Cumulative demand (gpm)	Assumed Supply (gpm)	Net Surplus (Deficit)	Cumulative Surplus (Deficit)
1	6	6100	0.638	10,792	10,792	10,792	0	0
2	7	10400	1.089	18,399	29,191	16,903	-89,771	-89,771
3	8	13500	1.413	23,884	53,075	16,903	-418,834	-508,605
4	9	14600	1.528	25,830	78,904	16,903	-535,598	-1,044,202
5	10	14600	1.528	25,830	104,734	16,903	-535,598	-1,579,800
6	11	13800	1.444	24,414	129,148	16,903	-450,678	-2,030,478
7	12	13800	1.444	24,414	153,562	16,903	-450,678	-2,481,156
8	13	12200	1.277	21,584	175,146	16,903	-280,840	-2,761,996
9	14	9100	0.952	16,099	191,245	16,903	48,223	-2,713,773
10	15	9100	0.952	16,099	207,345	16,903	48,223	-2,665,551
11	16	9400	0.984	16,630	223,975	16,903	16,378	-2,649,173
12	17	10200	1.068	18,045	242,020	16,903	-68,541	-2,717,714
13	18	13200	1.382	23,353	265,373	16,903	-386,989	-3,104,703
14	19	10300	1.078	18,222	283,595	16,903	-79,156	-3,183,859
15	20	12100	1.266	21,407	305,002	16,903	-270,225	-3,454,084
16	21	10200	1.068	18,045	323,047	16,903	-68,541	-3,522,625
17	22	9600	1.005	16,984	340,031	16,903	-4,852	-3,527,477
18	23	7700	0.806	13,622	353,654	16,903	196,832	-3,330,645
19	0	7700	0.806	13,622	367,276	16,903	196,832	-3,133,814
20	1	6800	0.712	12,030	379,306	16,903	292,366	-2,841,448
21	2	3400	0.356	6,015	385,321	16,903	653,273	-2,188,175
22	3	4300	0.450	7,607	392,929	16,903	557,739	-1,630,436
23	4	3900	0.408	6,900	399,828	16,903	600,198	-1,030,238
24	5	3300	0.345	5,838	405,667	16,903	663,888	-366,350
					Maximum Surplus (Deficit):			-3,527,477

Notes:
Determine Peak hour equalization requried based on dirurnal curve.
Assume Capacity equals maximum day demand.
Hourly ratios from prior water system study.

Appendix No. 2:
City of Portage - Northeast Storage Facility Preliminary Design Report
Project No. G04071

				28.66 Total - mgd **19,903** total - gpm				
Hour No.	Hour	Flow from report (gpm)	Ratio	Projected 2025 hourly demand (gpm)	Projected 2025 Cumulative demand (gpm)	Assumed Supply (gpm)	Net Surplus (Deficit)	Cumulative Surplus (Deficit)
1	6	6100	0.638	12,707	12,707	12,707	0	0
2	7	10400	1.089	21,665	34,372	19,903	-105,720	-105,720
3	8	13500	1.413	28,123	62,495	19,903	-493,186	-598,906
4	9	14600	1.528	30,414	92,909	19,903	-630,674	-1,229,580
5	10	14600	1.528	30,414	123,323	19,903	-630,674	-1,860,254
6	11	13800	1.444	28,747	152,070	19,903	-530,683	-2,390,937
7	12	13800	1.444	28,747	180,818	19,903	-530,683	-2,921,620
8	13	12200	1.277	25,414	206,232	19,903	-330,700	-3,252,320
9	14	9100	0.952	18,957	225,189	19,903	56,766	-3,195,555
10	15	9100	0.952	18,957	244,145	19,903	56,766	-3,138,789
11	16	9400	0.984	19,582	263,727	19,903	19,269	-3,119,520
12	17	10200	1.068	21,248	284,975	19,903	-80,722	-3,200,242
13	18	13200	1.382	27,498	312,473	19,903	-455,689	-3,655,931
14	19	10300	1.078	21,456	333,929	19,903	-93,221	-3,749,152
15	20	12100	1.266	25,206	359,135	19,903	-318,201	-4,067,354
16	21	10200	1.068	21,248	380,383	19,903	-80,722	-4,148,076
17	22	9600	1.005	19,998	400,382	19,903	-5,729	-4,153,804
18	23	7700	0.806	16,040	416,422	19,903	231,751	-3,922,054
19	0	7700	0.806	16,040	432,462	19,903	231,751	-3,690,303
20	1	6800	0.712	14,165	446,628	19,903	344,241	-3,346,062
21	2	3400	0.356	7,083	453,710	19,903	769,204	-2,576,859
22	3	4300	0.450	8,958	462,668	19,903	656,714	-1,920,145
23	4	3900	0.408	8,124	470,792	19,903	706,709	-1,213,436
24	5	3300	0.345	6,874	477,667	19,903	781,703	-431,733
		9554.167						
					Maximum Surplus (Deficit):			-4,153,804

Notes:
Determine Peak hour equalization requried based on dirurnal curve
Assume Capacity equals maximum day demand
Hourly ratios from prior water system study

Appendix No. 3: Elevated Storage Volume Calculation Data
City of Portage - Northeast Storage Facility Preliminary Design Report
Northeast Storage Facility Preliminary Design Report
Project No. G04071

1. Peak Hour Equalization				
Year	2005	2015	2025	
MDD	18.83	24.34	28.66	
PKHR	28.245	36.51	42.99	
Storage Volume Used - gallons	1,467,358	1,952,305	2,426,601	
2. Fire Flow				
Largest Fire Flow Demand - gpm	6000	6000	6000	
Duration - hours	4	4	4	
Volume - gallons	1,440,000	1,440,000	1,440,000	
3.a. 4 hour Emergency				
Year	2005	2015	2025	
ADD	5.9	7.62	8.97	
Duration of emergency	4	4	4	
Daily Volume - gallons	983,333	1,270,000	1,495,000	
3.b. 8 hour Emergency				
Year	2005	2015	2025	
ADD	5.9	7.62	8.97	
Duration of emergency	8	8	8	
Daily Volume - gallons	1,966,667	2,540,000	2,990,000	
3.c. 12 hour Emergency				
Year	2005	2015	2025	
ADD	5.9	7.62	8.97	
Duration of emergency	12	12	12	
Daily Volume - gallons	2,950,000	3,810,000	4,485,000	
Total				
1 + 2	2,907,358	3,392,305	3,866,601	Operational Storage + 4 hour 6000 gpm fire
1+ 2 + 3.a.	3,890,692	4,662,305	5,361,601	Operational Storage + 4 hour 6,000 gpm fire + 4 hour ADD reserve (1.25 hour MDD Reserve)
1+ 2 + 3.b.	4,874,025	5,932,305	6,856,601	Operational Storage + 4 hour 6,000 gpm fire + 8 hour ADD reserve (2.5 hour MDD Reserve)
1+ 2 + 3.c.	5,857,358	7,202,305	8,351,601	Operational Storage + 4 hour 6,000 gpm fire + 12 hour ADD reserve (3.75 hour MDD Reserve)
Compare to 25% MDD	4,707,500	6,085,000	7,165,000	

Appendix No. 3: Elevated Storage Volume Calculation Data - Peak Hour 2005
City of Portage - Northeast Storage Facility Preliminary Design Report
Northeast Storage Facility Preliminary Design Report
Project No. G04071

Demand	Supply
18.83 Total - mgd	21.23 Total - mgd
	'(includes 1 Shuman Pump)
13,076 Total - gpm	**14,743** Total - gpm

Hour No.	Hour	Flow from Report (gpm)	Ratio	Projected 2005 Hourly Demand (gpm)	Projected 2005 Cumulative Demand (gpm)	Assumed Supply (gpm)	Net Surplus (Deficit)	Cumulative Surplus (Deficit)
1	6	6100	0.638	8,349	8,349	8,349	0	0
2	7	10400	1.089	14,234	22,583	14,234	0	0
3	8	13500	1.413	18,477	41,060	14,743	-224,030	-224,030
4	9	14600	1.528	19,982	61,042	14,743	-314,361	-538,391
5	10	14600	1.528	19,982	81,025	14,743	-314,361	-852,752
6	11	13800	1.444	18,887	99,912	14,743	-248,666	-1,101,418
7	12	13800	1.444	18,887	118,800	14,743	-248,666	-1,350,084
8	13	12200	1.277	16,698	135,497	14,743	-117,274	-1,467,358
9	14	9100	0.952	12,455	147,952	14,743	137,296	-1,330,062
10	15	9100	0.952	12,455	160,407	14,743	137,296	-1,192,766
11	16	9400	0.984	12,865	173,272	14,743	112,660	-1,080,106
12	17	10200	1.068	13,960	187,232	14,743	46,964	-1,033,142
13	18	13200	1.382	18,066	205,299	14,743	-199,394	-1,232,536
14	19	10300	1.078	14,097	219,396	14,743	38,753	-1,193,783
15	20	12100	1.266	16,561	235,957	14,743	-109,063	-1,302,846
16	21	10200	1.068	13,960	249,917	14,743	46,964	-1,255,881
17	22	9600	1.005	13,139	263,056	14,743	96,236	-1,159,645
18	23	7700	0.806	10,539	273,595	14,743	252,263	-907,382
19	0	7700	0.806	10,539	284,133	14,743	252,263	-655,119
20	1	6800	0.712	9,307	293,440	14,743	326,171	-328,948
21	2	3400	0.356	4,653	298,094	10,136	328,954	6
22	3	4300	0.450	5,885	303,979	5,885	0	6
23	4	3900	0.408	5,338	309,317	5,338	0	6
24	5	3300	0.345	4,517	313,833	4,517	0	6
					Maximum Surplus (Deficit):			-1,467,358

Notes:
1. Determine Peak hour equalization requried based on dirurnal curve.
2. Assume max supply rate equals frim pump capacity + 2000 gpm (one shuman pump).
3. Hourly ratios from prior water system study.

Appendix No. 3: Elevated Storage Volume Calculation Data - Peak Hour 2015
City of Portage - Northeast Storage Facility Preliminary Design Report
Northeast Storage Facility Preliminary Design Report
Project No. G04071

Demand	Supply
24.34 Total - mgd	27.22 Total - mgd
	(Includes one Shuman Pump)
16,903 total - gpm	**18,903** total - gpm

Hour No.	Hour	Flow from report (gpm)	Ratio	Projected 2015 hourly demand (gpm)	Projected 2015 Cumulative demand (gpm)	Assumed Supply (gpm)	Net Surplus (Deficit)	Cumulative Surplus (Deficit)
1	6	6100	0.638	10,792	10,792	10,792	0	0
2	7	10400	1.089	18,399	29,191	18,399	0	0
3	8	13500	1.413	23,884	53,075	18,903	-298,847	-298,847
4	9	14600	1.528	25,830	78,904	18,903	-415,611	-714,458
5	10	14600	1.528	25,830	104,734	18,903	-415,611	-1,130,069
6	11	13800	1.444	24,414	129,148	18,903	-330,692	-1,460,760
7	12	13800	1.444	24,414	153,562	18,903	-330,692	-1,791,452
8	13	12200	1.277	21,584	175,146	18,903	-160,853	-1,952,305
9	14	9100	0.952	16,099	191,245	18,903	168,209	-1,784,095
10	15	9100	0.952	16,099	207,345	18,903	168,209	-1,615,886
11	16	9400	0.984	16,630	223,975	18,903	136,365	-1,479,521
12	17	10200	1.068	18,045	242,020	18,903	51,445	-1,428,076
13	18	13200	1.382	23,353	265,373	18,903	-267,002	-1,695,078
14	19	10300	1.078	18,222	283,595	18,903	40,830	-1,654,248
15	20	12100	1.266	21,407	305,002	18,903	-150,238	-1,804,486
16	21	10200	1.068	18,045	323,047	18,903	51,445	-1,753,040
17	22	9600	1.005	16,984	340,031	18,903	115,135	-1,637,906
18	23	7700	0.806	13,622	353,654	18,903	316,818	-1,321,087
19	0	7700	0.806	13,622	367,276	18,903	316,818	-1,004,269
20	1	6800	0.712	12,030	379,306	18,903	412,352	-591,917
21	2	3400	0.356	6,015	385,321	15,881	591,923	6
22	3	4300	0.450	7,607	392,929	7,607	0	6
23	4	3900	0.408	6,900	399,828	6,900	0	6
24	5	3300	0.345	5,838	405,667	5,838	0	6
					Maximum Surplus (deficit)			-1,952,305

Notes:
1. Determine Peak hour equalization requried based on dirurnal curve
2. Hourly ratios from prior water system study

Appendix No. 3: Elevated Storage Volume Calculation Data - Peak Hour 2025
City of Portage - Northeast Storage Facility Preliminary Design Report
Northeast Storage Facility Preliminary Design Report
Project No. G04071

Demand	Supply
28.66 Total - mgd	31.54 Total - mgd
	(includes 1 Shuman pump)
19,903 total - gpm	**21,903** total - gpm

Hour No.	Hour	Flow from Report (gpm)	Ratio	Projected 2025 Hourly Demand (gpm)	Projected 2025 Cumulative Demand (gpm)	Assumed Supply (gpm)	Net Surplus (Deficit)	Cumulative Surplus (Deficit)
1	6	6100	0.638	12,707	12,707	12,707	0	0
2	7	10400	1.089	21,665	34,372	21,665	0	0
3	8	13500	1.413	28,123	62,495	21,903	-373,186	-373,186
4	9	14600	1.528	30,414	92,909	21,903	-510,674	-883,860
5	10	14600	1.528	30,414	123,323	21,903	-510,674	-1,394,534
6	11	13800	1.444	28,747	152,070	21,903	-410,683	-1,805,217
7	12	13800	1.444	28,747	180,818	21,903	-410,683	-2,215,900
8	13	12200	1.277	25,414	206,232	21,903	-210,700	-2,426,601
9	14	9100	0.952	18,957	225,189	21,903	176,766	-2,249,835
10	15	9100	0.952	18,957	244,145	21,903	176,766	-2,073,069
11	16	9400	0.984	19,582	263,727	21,903	139,269	-1,933,800
12	17	10200	1.068	21,248	284,975	21,903	39,278	-1,894,522
13	18	13200	1.382	27,498	312,473	21,903	-335,689	-2,230,211
14	19	10300	1.078	21,456	333,929	21,903	26,779	-2,203,432
15	20	12100	1.266	25,206	359,135	21,903	-198,201	-2,401,634
16	21	10200	1.068	21,248	380,383	21,903	39,278	-2,362,356
17	22	9600	1.005	19,998	400,382	21,903	114,271	-2,248,084
18	23	7700	0.806	16,040	416,422	21,903	351,751	-1,896,334
19	0	7700	0.806	16,040	432,462	21,903	351,751	-1,544,583
20	1	6800	0.712	14,165	446,628	21,903	464,241	-1,080,342
21	2	3400	0.356	7,083	453,710	21,903	889,204	-191,139
22	3	4300	0.450	8,958	462,668	12,143	191,127	-12
23	4	3900	0.408	8,124	470,792	8,124	0	-12
24	5	3300	0.345	6,874	477,667	6,874	0	-12
						Maximum Surplus (Deficit)		-2,426,601

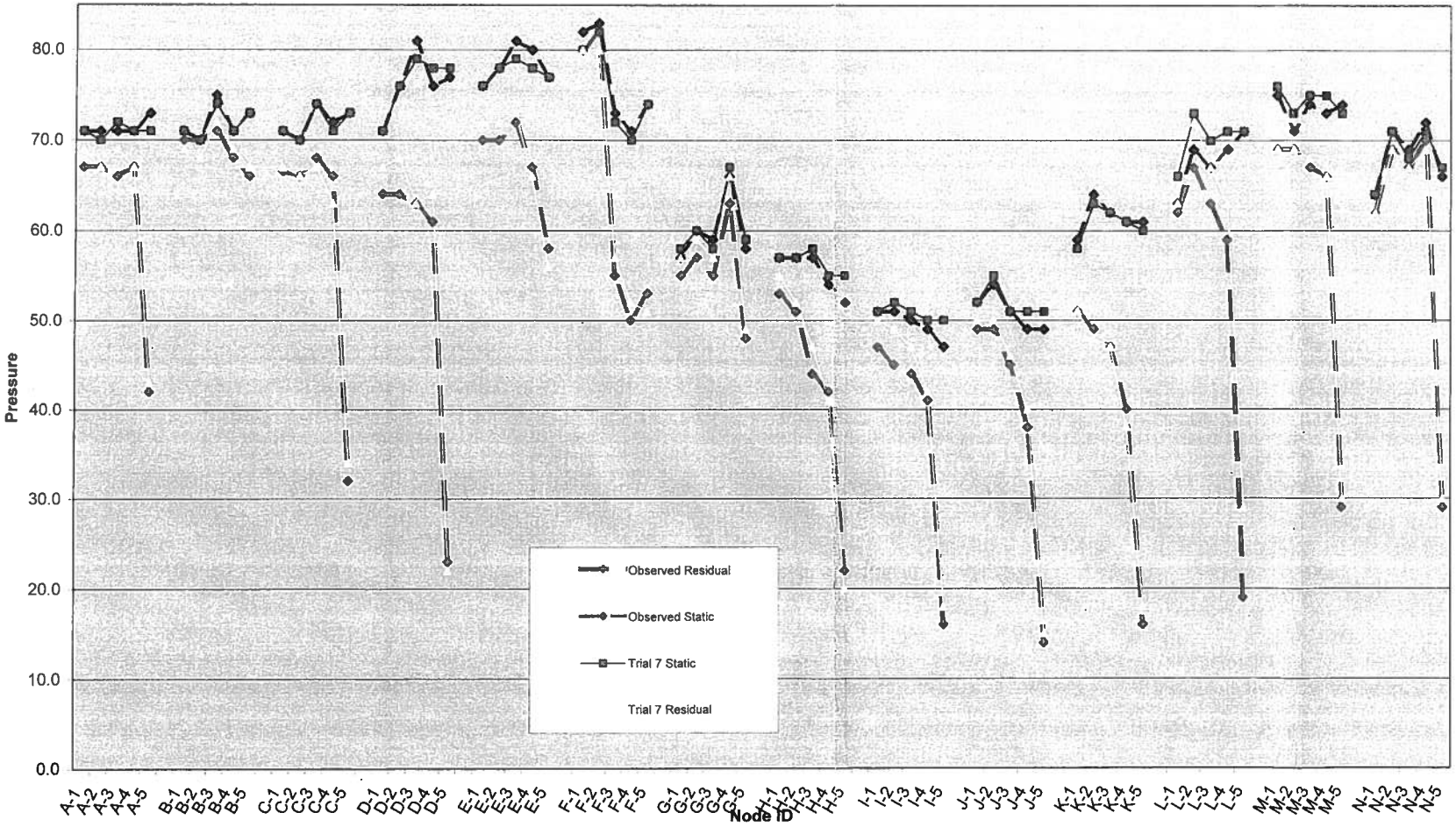
Notes:
1. Determine Peak hour equalization requried based on dirurnal curve
2. Hourly ratios from prior water system study

Appendix No. 4
City of Portage Hydraulic Model
Calibration Spreadsheet
Project No. G04071

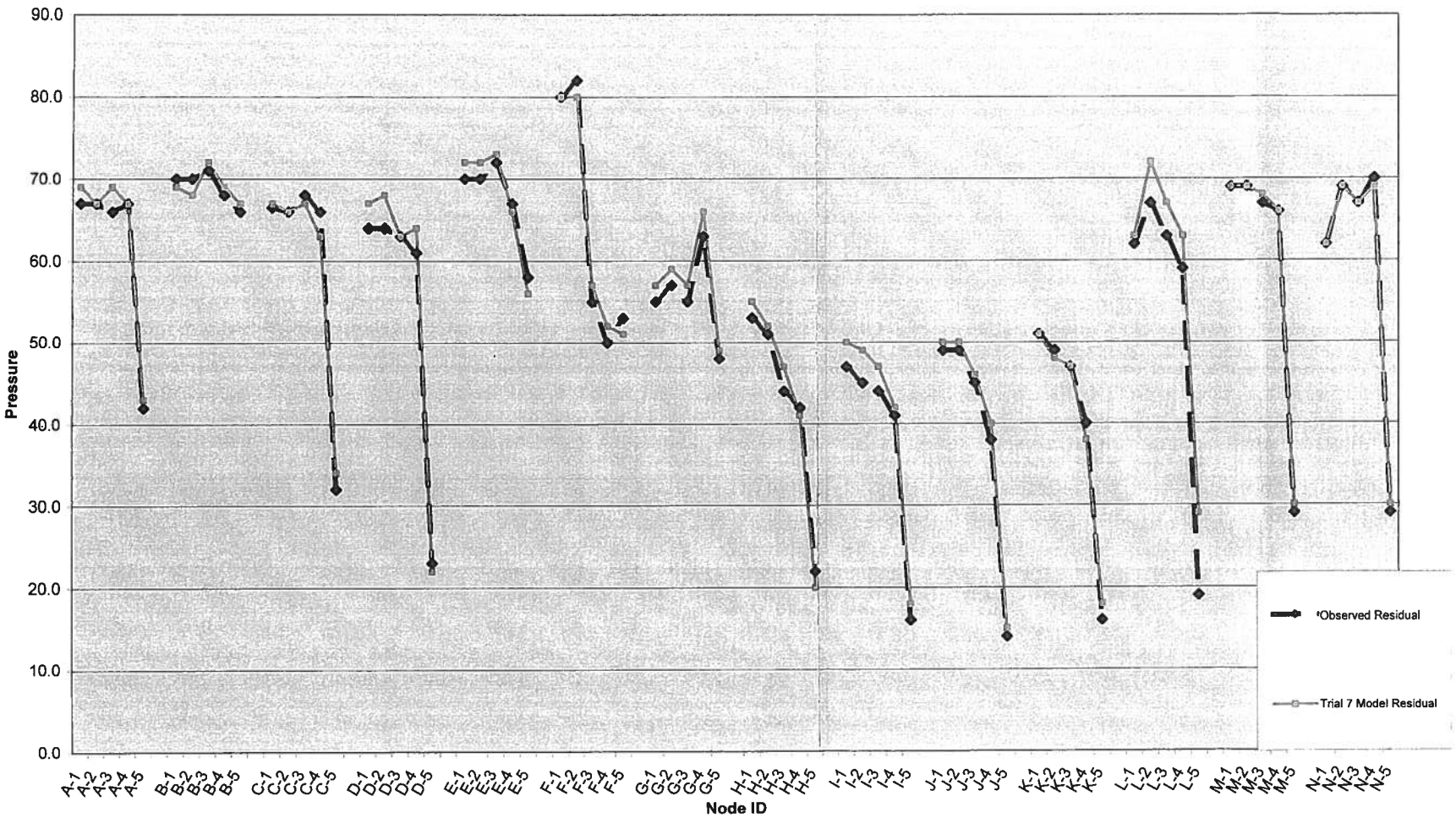
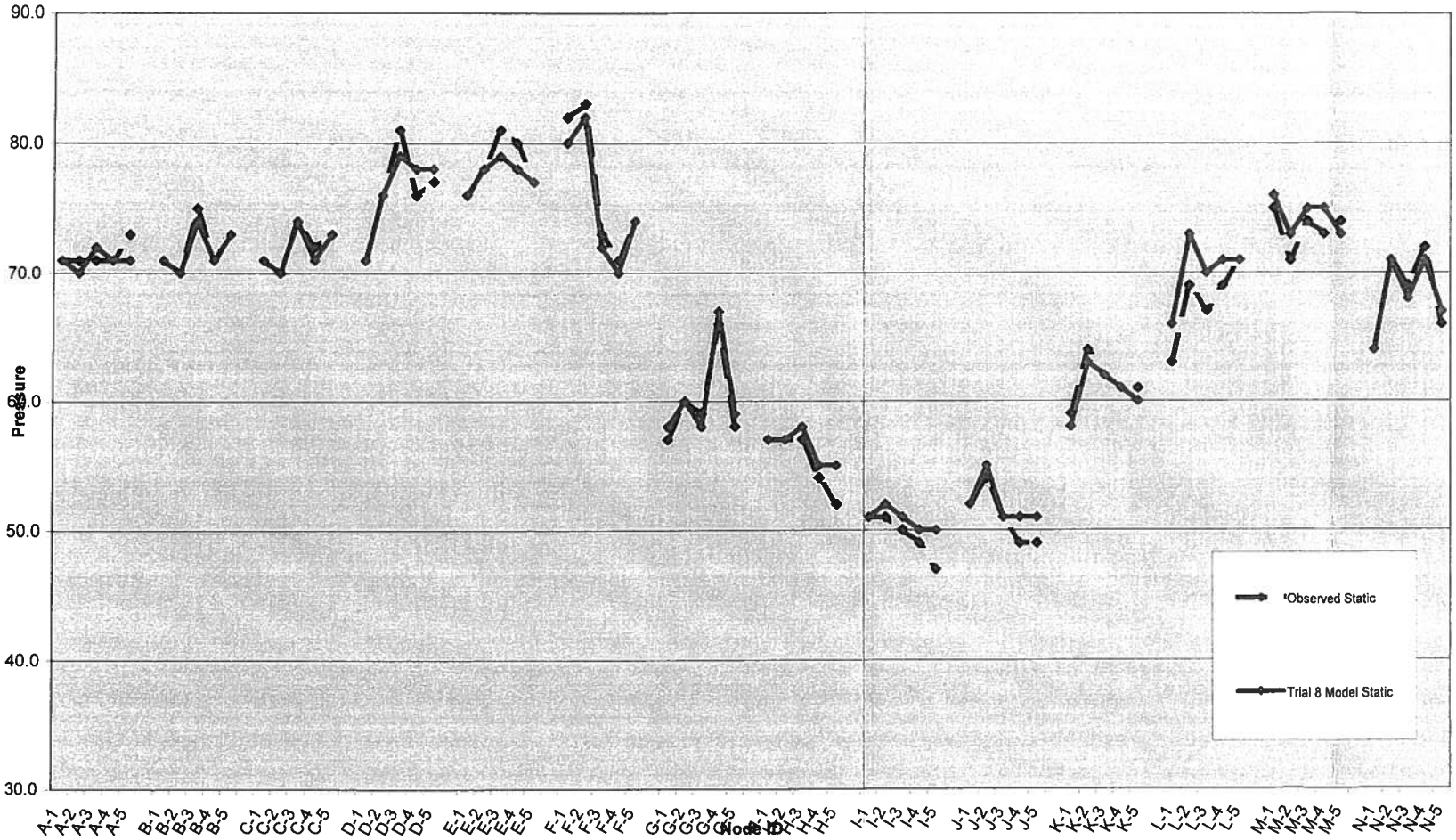
Model Node ID	Field Observed Static Hyd Pressure (psi)	Adjust for Gauge Discrepancies	Adjusted Field Observed Static Hyd Pressure (psi)	Trial 1 Model Static Pressure (PSI)	Static Pressure difference (PSI)	Field Observed Hyd Flow (gpm)	Field Observed Residual Hyd Pressure (psi)	Adjusted Observed Residual Hyd Pressure (psi)	Trial 1 Model Residual Pressure (PSI)	Trial 1 Residual Pressure difference (PSI)	Trial 1 Model Residual Pressure (PSI)	Trial 1 Residual Pressure difference (PSI)	Trial 2 Model Static Pressure (PSI)	Trial 2 Static Pressure difference (PSI)	Trial 2 Model Residual Pressure (PSI)	Trial 2 Residual Pressure difference (PSI)	Trial 3 Model Static Pressure (PSI)	Trial 3 Static Pressure difference (PSI)	Trial 3 Model Residual Pressure (PSI)	Trial 3 Residual Pressure difference (PSI)	Trial 4 Model Static Pressure (PSI)	Trial 4 Static Pressure difference (PSI)	Trial 4 Model Residual Pressure (PSI)	Trial 4 Residual Pressure difference (PSI)	Trial 5 Model Static Pressure (PSI)	Trial 5 Static Pressure difference (PSI)	Trial 5 Model Residual Pressure (PSI)	Trial 5 Residual Pressure difference (PSI)	Trial 6 Model Static Pressure (PSI)	Trial 6 Static Pressure difference (PSI)	Trial 6 Model Residual Pressure (PSI)	Trial 6 Residual Pressure difference (PSI)	Trial 7 Model Static Pressure (PSI)	Trial 7 Static Pressure difference (PSI)	Trial 7 Model Residual Pressure (PSI)	Trial 7 Residual Pressure difference (PSI)
A-1	71.0	0.0	71.0	71.0	0.0		67.0	67.0	69	-2.0	69	-2.0		71.0	69	-2.0		71.0	69	-2.0	71	0.0	69	-2.0		71.0	69	-2.0		71.0		67.0	71	0.0	69	-2.0
A-2	71.0	0.0	71.0	71.0	0.0		67.0	67.0	67	0.0	67	0.0		71.0	67	0.0		71.0	67	0.0	71	0.0	67	0.0		71.0	67	0.0		71.0		67.0	70	1.0	67	0.0
A-3	71.0	0.0	71.0	72.0	-1.0		66.0	66.0	69	-3.0	69	-3.0		71.0	69	-3.0		71.0	69	-3.0	72	-1.0	69	-3.0		71.0	69	-3.0		71.0		66.0	72	-1.0	69	-3.0
A-4	71.0	0.0	71.0	72.0	-1.0		67.0	67.0	67	0.0	67	0.0		71.0	67	0.0		71.0	67	0.0	71	0.0	68	-1.0		71.0	67	0.0		71.0		67.0	71	0.0	67	0.0
A-5	71.0	-2.0	73.0	71.0	2.0	2780.0	42.0	42.0	24	18.0	24	18.0		73.0	26	16.0		73.0	44	-2.0	72	1.0	44	-2.0		73.0	44	-2.0		73.0		42.0	71	2.0	43	-1.0
B-1	71.0	0.0	71.0	71.0	0.0		70.0	70.0	69	1.0	69	1.0		71.0	70	0.0		71.0	70	0.0	71	0.0	70	0.0		71.0	70	0.0		71.0		70.0	71	0.0	69	1.0
B-2	70.0	0.0	70.0	70.0	0.0		70.0	70.0	68	2.0	68	2.0		70.0	69	1.0		70.0	69	1.0	70	0.0	69	1.0		70.0	69	1.0		70.0		70.0	70	0.0	68	2.0
B-3	75.0	0.0	75.0	74.0	1.0		71.0	71.0	72	-1.0	72	-1.0		75.0	72	-1.0		75.0	72	-1.0	74	1.0	72	-1.0		75.0	72	-1.0		75.0		71.0	74	1.0	72	-1.0
B-4	71.0	0.0	71.0	71.0	0.0		68.0	68.0	69	-1.0	69	-1.0		71.0	69	-1.0		71.0	69	-1.0	71	0.0	69	-1.0		71.0	69	-1.0		71.0		68.0	71	0.0	69	-1.0
B-5	71.0	-2.0	73.0	73.0	0.0	1080.0	66.0	66.0	65	1.0	65	1.0		73.0	65	1.0		73.0	67	-1.0	74	-1.0	67	-1.0		73.0	67	-1.0		73.0		66.0	73	0.0	67	-1.0
C-1	71.0	0.0	71.0	71.0	0.0		66.5	66.5	66	0.5	66	0.5		71.0	67	-0.5		71.0	67	-0.5	71	0.0	67	-0.5		71.0	67	-0.5		71.0		66.5	71	0.0	67	-0.5
C-2	70.0	0.0	70.0	70.0	0.0		66.0	66.0	64	2.0	64	2.0		70.0	66	0.0		70.0	66	0.0	70	0.0	66	0.0		70.0	66	0.0		70.0		66.0	70	0.0	66	0.0
C-3	74.0	0.0	74.0	74.0	0.0		68.0	68.0	65	3.0	65	3.0		74.0	68	0.0		74.0	68	0.0	74	0.0	68	0.0		74.0	67	1.0		74.0		68.0	74	0.0	67	1.0
C-4	72.0	0.0	72.0	71.0	1.0		66.0	66.0	62	4.0	62	4.0		72.0	64	2.0		72.0	64	2.0	71	1.0	64	2.0		72.0	64	2.0		72.0		66.0	71	1.0	63	3.0
C-5	71.0	-2.0	73.0	73.0	0.0	2430.0	32.0	32.0	34	-2.0	34	-2.0		73.0	37	-5.0		73.0	34	-2.0	73	0.0	34	-2.0		73.0	34	-2.0		73.0		32.0	73	0.0	34	-2.0
D-1	71.0	0.0	71.0	71.0	0.0		64.0	64.0	67	-3.0	67	-3.0		71.0	68	-4.0		71.0	68	-4.0	71	0.0	68	-4.0		71.0	68	-4.0		71.0		64.0	71	0.0	67	-3.0
D-2	76.0	0.0	76.0	76.0	0.0		64.0	64.0	67	-3.0	67	-3.0		76.0	68	-4.0		76.0	68	-4.0	76	0.0	68	-4.0		76.0	68	-4.0		76.0		64.0	76	0.0	68	-4.0
D-3	81.0	0.0	81.0	79.0	2.0		63.0	63.0	60	3.0	60	3.0		81.0	63	0.0		81.0	63	0.0	79	2.0	64	-1.0		81.0	64	-1.0		81.0		63.0	79	2.0	63	0.0
D-4	76.0	0.0	76.0	78.0	-2.0		61.0	61.0	60	1.0	60	1.0		76.0	63	-2.0		76.0	64	-3.0	78	-2.0	64	-3.0		76.0	64	-3.0		76.0		61.0	78	-2.0	64	-3.0
D-5	75.0	-2.0	77.0	78.0	-1.0	2065.0	23.0	23.0	27	-4.0	27	-4.0		77.0	32	-9.0		77.0	23	0.0	78	-1.0	23	0.0		77.0	22	1.0		77.0		23.0	78	-1.0	22	1.0
E-1	76.0	0.0	76.0	76.0	0.0		70.0	70.0	72	-2.0	72	-2.0		76.0	72	-2.0		76.0	72	-2.0	76	0.0	72	-2.0		76.0	72	-2.0		76.0		70.0	76	0.0	72	-2.0
E-2	78.0	0.0	78.0	77.0	1.0		70.0	70.0	71	-1.0	71	-1.0		78.0	71	-1.0		78.0	72.00	-2.0	77	1.0	72	-2.0		78.0	72	-2.0		78.0		70.0	78	0.0	72	-2.0
E-3	81.0	0.0	81.0	79.0	2.0		72.0	72.0	72	0.0	72	0.0		81.0	72	0.0		81.0	73.00	-1.0	79	2.0	73	-1.0		81.0	73	-1.0		81.0		72.0	79	2.0	73	-1.0
E-4	80.0	0.0	80.0	78.0	2.0		67.0	67.0	63	4.0	63	4.0		80.0	61	6.0		80.0	66.00	1.0	78	2.0	66	1.0		80.0	66	1.0		80.0		67.0	78	2.0	66	1.0
E-5	75.0	-2.0	77.0	77.0	0.0	1020.0	58.0	58.0	48	10.0	48	10.0		77.0	42	16.0		77.0	56.00	2.0	77	0.0	57	1.0		77.0	56	2.0		77.0		58.0	77	0.0	56	2.0
F-1	82.0	0.0	82.0	80.0	2.0		80.0	80.0	80	0.0	80	0.0		82.0	80	0.0		82.0	80	0.0	80	2.0	80	0.0		82.0	80	0.0		82.0		80.0	80	2.0	80	0.0
F-2	83.0	0.0	83.0	82.0	1.0		82.0	82.0	79	3.0	79	3.0		83.0	80	2.0		83.0	80	2.0	82	1.0	80	2.0		83.0	80	2.0		83.0		82.0	82	1.0	80	2.0
F-3	73.0	0.0	73.0	72.0	1.0		55.0	55.0	56	-1.0	56	-1.0		73.0	58	-3.0		73.0	56	-1.0	72	1.0	57	-2.0		73.0	57	-2.0		73.0		55.0	72	1.0	57	-2.0
F-4	71.0	0.0	71.0	70.0	1.0		50.0	50.0	52	-2.0	52	-2.0		71.0	54	-4.0		71.0	51	-1.0	70	1.0	52	-2.0		71.0	52	-2.0		71.0		50.0	70	1.0	52	-2.0
F-5	72.0	-2.0	74.0	74.0	0.0	940.0	53.0	53.0	49	4.0	49	4.0		74.0	51	2.0		74.0	50	3.0	74	0.0	51	2.0		74.0	51	2.0		74.0		53.0	74	0.0	51	2.0
G-1	57.0	0.0	57.0	58.0	-1.0		55.0	55.0	57	-2.0	57	-2.0		57.0	57	-2.0		57.0	57	-2.0	58	-1.0	58	-3.0		57.0	58	-3.0		57.0		55.0	58	-1.0	57	-2.0
G-2																					60												60			
G-3	59.0	0.0	59.0	58.0	1.0		55.0	55.0	57	-2.0	57	-2.0		59.0	57	-2.0		59.0	57	-2.0	59	0.0	57	-2.0		59.0	57	-2.0		59.0		55.0	58	1.0	57	-2.0
G-4	66.0	0.0	66.0	67.0	-1.0		63.0	63.0	66	-3.0	66	-3.0		66.0	65	-2.0		66.0	66	-3.0	67	-1.0	66	-3.0		66.0	66	-3.0		66.0		63.0	67	-1.0	66	-3.0
G-5																					59												59			
H-1	57.0	0.0	57.0	57.0	0.0		53.0	53.0	55	-2.0	55	-2.0		57.0	55	-2.0		57.0	55	-2.0	57	0.0	55	-2.0		57.0	55	-2.0		57.0		53.0	57	0.0	55	-2.0
H-2	57.0	0.0	57.0	57.0	0.0		51.0	51.0	52	-1.0	52	-1.0		57.0	52	-1.0		57.0	52	-1.0	57	0.0	52	-1.0		57.0	52	-1.0		57.0		51.0	57	0.0	52	-1.0
H-3	57.0	0.0	57.0	58.0	-1.0		44.0	44.0	47	-3.0	47	-3.0		57.0	47	-3.0		57.0	47	-3.0	58	-1.0	48	-4.0		57.0	48	-4.0		57.0		44.0	58	-1.0	47	-3.0
H-4	54.0	0.0	54.0	55.0	-1.0		42.0	42.0	41	1.0	41	1.0		54.0	40	2.0		54.0	42	0.0	55	-1.0	41	1.0		54.0	41	1.0		54.0		42.0	55	-1.0	41	1.0
H-5																					55												55			
I-1	51.0	0.0	51.0	51.0	0.0		47.0	47.0	50	-3.0	50	-3.0		51.0	50	-3.0		51.0	50	-3.0	51	0.0	50	-3.0		51.0	50	-3.0		51.0		47.0	51	0.0	50	-3.0
I-2	51.0	0.0	51.0	52.0	-1.0		45.0	45.0	49	-4.0	49	-4.0		51.0	49	-4.0		51.0	49	-4.0	52	-1.0	49	-4.0		51.0	49	-4.0		51.0		45.0	52	-1.0	49	-4.0
I-3	50.0	0.0	50.0	51.0	-1.0		44.0	44.0	46	-2.0	46	-2.0		50.0	47	-3.0		50.0	46	-2.0	51	-1.0	47	-3.0		50.0	47	-3.0		50.0		44.0	51	-1.0	47	-3.0
I-4	49.0	0.0	49.0	50.0	-1.0		41.0	41.0	42	-1.0	42	-1.0		49.0	43	-2.0		49.0	42	-1.0	50	-1.0	42	-1.0		49.0	42	-1.0		49.0		41.0	50	-1.0	42	-1.0
I-5																				-4.0	50												50			
J-1	52.0	0.0	52.0	52.0	0.0		49.0	49.0	50	-1.0	50	-1.0		52.0	50	-1.0		52.0	50	-1.0	52	0.0	50	-1.0		52.0	50	-1.0		52.0		49.0	52	0.0	50	-1.0
J-2	54.0	0.0	54.0	54.0	0.0		49.0	49.0	50	-1.0	50	-1.0		54.0	50	-1.0		54.0	50	-1.0	55	-1.0	50	-1.0		54.0	50	-1.0		54.0		49.0	55	-1.0	50	-1.0
J-3	51.0	0.0	51.0	51.0	0.0		45.0	45.0	46	-1.0	46	-1.0		51.0	46	-1.0		51.0	46	-1.0	51	0.0	46	-1.0		51.0	46	-1.0		51.0		45.0	51	0.0	46	-1.0
J-4	49.0	0.0	49.0	50.0	-1.0		38.0	38.0	40	-2.0	40	-2.0		49.0	40	-2.0		49.0	39	-1.0	51	-2.0	40	-2.0		49.0	40	-2.0		49.0		38.0	51	-2.0	40	-2.0
J-5																					51												51			
K-1	59.0	0.0	59.0	58.0	1.0		51.0	51.0	49	2.0	49	2.0		59.0	52	-1.0		59.0	51	0.0	58	1.0	51	0.0		59.0	51	0.0		59.0		51.0	58	1.0	51	0.0
K-2	64.0	0.0	64.0	63.0	1.0		49.0	49.0	45	4.0	45	4.0		64.0	51	-2.0		64.0	48	1.0	62	2.0	48	1.0		64.0	48	1.0		64.0		49.0	63	1.0	48	1.0
K-3	62.0	0.0	62.0	62.0	0.0		47.0	47.0	43	4.0	43	4.0		62.0	49	-2.0		62.0	47	0.0	62	0.0	47	0.0		62.0	47	0.0		62.0		47.0	62	0.0	47	0.0
K-4	61.0	0.0	61.0	61.0	0.0		40.0	40.0	33	7.0	33	7.0		61.0	42	-2.0		61.0	39	1.0	62	-1.0	39	1.0		61.0	39	1.0		61.0		40.0	61	0.0	38	2.0
K-5																					61												60			
L-1	63.0	0.0	63.0	66.0	-3.0		62.0	62.0	64	-2.0	64	-2.0		63.0	64	-2.0		63.0	64	-2.0	66	-3.0	64	-2.0		63.0	64	-2.0		63.0	64	-2.0	66	-3.0	63	-1.0
L-2	69.0	0.0	69.0	73.0	-4.0		67.0	67.0	72	-5.0	72	-5.0		69.0	72	-5.0		69.0	72	-5.0	73	-4.0	72	-5.0		69.0	72	-5.0		69.0	72	-5.0	73	-4.0	72	-5.0
L-3	67.0	0.0	67.0	70.0	-3.0		63.0	63.0	68	-5.0	68	-5.0		67.0	68	-5.0		67.0	68	-5.0	70	-3.0	68	-5.0		67.0	68	-5.0		67.0	68	-5.0	70	-3.0	67	-4.0
L-4	69.0	0.0	69.0	73.0	-4.0		59.0	59.0	65	-6.0	65	-6.0		69.0	66	-7.0		69.0	66	-7.0	73	-4.0	66	-7.0		69.0	66	-7.0		69.0	66	-7.0	71	-2.0	63	-4.0
L-5	69.0	-2.0	71.0	73.0	-2.0	1870.0	19.0	19.0	51	-32.0	51	-32.0		71.0	51	-32.0		71.0	52	-33.0	73	-2.0	52	-33.0		71.0	52	-33.0		71.0	50	-31.0	71	0.0	29	-10.0
M-1	75.0	0.0	75.0	75.0	0.0		69.0	69.0	65	4.0	65	4.0		75.0	68	1.0		75.0	65	4.0	75	0.0	67	2.0		75.0	66	3.0		75.0	68	1.0	76	-1.0	69	0.0
M-2	71.0	0.0	71.0	72.0	-1.0		69.0	69.0	67	2.0	67	2.0		71.0	68	1.0		71.0	68	1.0	72	-1.0	68	1.0		71.0	68	1.0		71.0	69	0.0	73	-2.0	69	0.0
M-3	74.0	0.0	74.0	75.0	-1.0		67.0	67.0	65	2.0	65	2.0		74.0	67	0.0		74.0	65	2.0	75	-1.0	66	1.0		74.0	63	4.0		74.0	67	0.0	75	-1.0	68	-1.0
M-4	73.0	0.0	73.0	73.0	0.0		66.0	66.0	61	5.0	61	5.0		73.0	65	1.0		73.0	60	6.0	73	0.0	62	4.0		73.0	62	4.0		73.0	64	2.0	75	-2.0	66	0.0
M-5	72.0	-2.0	74.0	73.0	1.0	2316.0	29.0	29.0	36	-7.0	36	-7.0		74.0	40	-11.0		74.0	25	4.0	72	2.0	33	-4.0		74.0	33	-4.0		74.0	35	-6.0	73	1.0	30	-1.0
N-1	64.0	0.0	64.0	64.0	0.0		62.0	62.0	62	0.0	62	0.0		64.0	63	-1.0		64.0	62	0.0	64	0.0	63	-1.0		64.0	63	-1.0		64.0		62.0	64	0.0	62	0.0
N-2	71.0	0.0	71.0	71.0	0.0		69.0	69.0	69	0.0	69	0.0		71.0	69	0.0		71.0	69	0.0	71	0.0	69	0.0		71.0	69	0.0		71.0		69.0	71	0.0	69	0.0
N-3	69.0	0.0	69.0	68.0	1.0		67.0	67.0	67	0.0	67	0.0		69.0	67	0.0		69.0	67	0.0	68	1.0	67	0.0		69.0	67	0.0		69.0		67.0	68	1.0	67	0.0
N-4	72.0	0.0	72.0	71.0	1.0		70.0	70.0	69	1.0	69	1.0		72.0	69	1.0		72.0	69	1.0	71	1.0	69	1.0		72.0	69	1.0		72.0		70.0	71	1.0	69	1.0
N-5																					67												67			

Notes:
Measurements based on May 2004 Flow Tests.

City of Portage Reliability Study
Calibration Summary - Static and Residual Pressures



City of Portage Reliability Study
Calibration Summary - Residual Pressures



City of Portage Reliability Study
Calibration Summary - Static Pressures