

City of Logan, Utah

Culinary Water System Master Plan



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April 2007



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October 15, 2007

Mr. Lance E. Houser, P.E.
Water and Waste Water Division Manager
City of Logan
950 West 600 North
Logan, UT 84321

Dear Mr. Houser:

Enclosed is our final report for the City of Logan's Culinary Water System Master Plan.

We would like to extend our heartfelt gratitude for the contributions and insights provided by City staff during preparation of the master plan. This report would not have been possible without the efforts of the Water and Waste Water Division, City Engineer, and GIS and Community Development Departments.

We have enjoyed working with you in this planning exercise, and hope to be of service to the City in the future as opportunity may arise.

Very truly yours,

BLACK & VEATCH CORPORATION

A handwritten signature in black ink, appearing to read "Donald Champenois". A small "ALH" is written below the signature.

Donald Champenois

EDC:ah



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Water Conservation Plan



EXECUTIVE SUMMARY

Study Purpose

The City of Logan commissioned the Culinary Water System Master Plan in response to growing concern over shortcomings of the existing water supply system and its ability to meet future water demands and regulatory requirements. The Master Plan provides an update on water supply system status, and maps out prioritized improvements to ensure that the City can meet an acceptable level of service in the future.

Primary components of the project include:

- Development of a completely new GIS-based hydraulic model in H2OMAP™ for the existing water distribution system. This represented a major change in scope, as well as increased level of effort, where the City GIS Division played a key role. At the same time, GIS-based modeling offered greatest value to the City in planning for future water system development. The model was validated by SCADA data for operational parameters such as pressure, flow, and reservoir level.
- Vulnerability analyses to evaluate existing system response to supply emergencies. These hydraulic analyses highlighted major shortcomings with configuration of the current water supply system, and helped develop options for future improvements.
- Detailed evaluation of population and water demand growth through 2025 and their spatial distribution through the existing and future service area using GIS data.
- Development and evaluation of alternatives to meet City level of service objectives and respond to anticipated growth, particularly on the west side of the city. These evaluations included vulnerability analyses to evaluate various measures designed to provide operational flexibility and redundancy.
- A review of the existing City GIS, tools available to integrate the City GIS with the hydraulic model, and procedures for maintaining the GIS.
- Analysis of pipe failure patterns in order to develop a prioritized pipe replacement plan based on realistic pipe failure rates.
- Regulatory review to incorporate the requirements of existing, pending and future drinking water regulations into planned improvements as required.
- Preparation of the City of Logan draft Capital Improvements Plan through the year 2025, with investments in five-year increments.

Separate to the present report, Black & Veatch prepared the Water Conservation Plan, which the City of Logan submitted to the State of Utah Division of Drinking Water. In response to national security rules, a public version of the plan was also prepared.



The Water Conservation Plan addressed water rights and supply capacity, and water demand projections with and without water conservation. The plan also evaluated measures to reduce unaccounted-for water and significantly reduce unit consumption. The plan concluded that residential consumption was already quite low, and that the most cost-effective reduction in demand could be obtained by reducing unaccounted-for water.

Demographics and Water Use Projections

The annual average rate of change in population in Logan over the past 40 years has consistently exceeded projections developed by Cache County and the Utah Government Office of Planning and Budget.

The Master Plan employed population and employment data developed by the City Community Development Department. The Master Plan considered an annual rate of growth in population of about 2.1% as most realistic based on historic growth rates.

Projected development in western annexation areas was then added to obtain the growth projections shown in Table ES.1.

Table ES.1		
Logan Projected Population and Employment, 2005-2025		
Year	Population	Employment
2005	47,235	37,600
2010	52,226	44,847
2015	58,775	51,142
2020	66,146	58,320
2025	74,441	66,506

Evaluation of current unit water consumption showed that commercial and USU billed consumption amounted to about 50% of total consumption, underlying Logan's role as the economic hub of a larger metropolitan area. Residential billed consumption was only 100 gpcd, well below rates and even goals in other major Utah cities.



More importantly, evaluation of water supply and billed consumption revealed that unaccounted-for water comprised 42% of total consumption. Three main sources for water losses from the supply system were identified:

- DeWitt Springs Pipeline, for which a project is already underway to replace the pipe reaches in worst condition.
- Golf Course Tanks, for which a project was already underway to replace two tanks that were cracked and leaking. This project was completed in 2006.
- Leakage in the water distribution network, where the general age and failure data suggested problems not only in the center of town where the oldest pipes are located, but also in many newer areas of town.

The replacement of the damaged Golf Course tanks, completed in 2006, and the planned replacement of sections of the DeWitt Springs Pipeline, will result in an anticipated 75% reduction in the leakage component of unaccounted-for water.

City production records and SCADA production data were used to determine current seasonal and annual peak demands. Together with average annual water demand projection, these were used to establish the demands which could be anticipated through the planning period. Table ES.2 shows the demand projections used in the Master Plan.

Table ES.2			
Logan Water Demand Projections, mgd			
Category	2003-2005	2010	2025
Annual Average Day			
	Residential	4.50	5.22
	Industrial-Commercial-Institutional	4.48	4.70
	Unaccounted-for-Water	5.66	1.70
	Total	14.63 ¹	11.62
Maximum Day	26.60 ¹	20.85	29.29
Peak Hour	33.18 ²	27.77	39.02

¹Average of 2003-2004.

²Average of 2004-2005.



Existing Distribution System Facilities

Evaluation of the existing distribution system led to a series of key observations:

Large sections of the city experience supply pressures double that recommended by the Utah Division of Drinking Water. Pressures for the main zone and parts of the Cliffside sub-zone are about 190-220 psi. High pressures lead to reduced pipe life and increased leakage losses, and more importantly expose both the general public and City maintenance staff to a serious risk to life and property.

The groundwater supply wells provide some level of backup should the DeWitt Springs Pipeline fail. However, with the exception of the Willow Park well, the well screens and well pumps are quite old and their remaining useful lifetime in doubt. None of the wells is equipped with modern variable frequency controls, and there is a need to integrate the wells more fully into a City-wide supervision and control system. In the future, the need is foreseen for an additional well, and for new pumps and new pump configurations.

With the replacement of the Golf Course 1 and 2 tanks by newer cells, the condition of storage facilities is good, and the volume is also sufficient for operational purposes. However, the fact that all City storage is located on or adjacent to the earthquake fault line should give cause for concern. In the future additional storage will be required for operational purposes, and this should be located farther from the fault line. It should also be noted that the Utah Division of Drinking Water may require more storage.

One booster station has been decommissioned, but the others are reportedly in acceptable condition. However, hydraulic modeling suggested that some pumps may be oversized.

There are three operating PRV stations serving USU. Another six stations intended to lower pressure in the lower part of the main zone are not operated. However, in future planning these are incorporated into a plan to establish pressure zoning.

Hydraulic modeling of the distribution system confirms good delivery capacity due to a series of large mains. However, one-third of the total network consists of pipes too small to provide fire flow capacity per State of Utah requirements. Pipe condition was further evaluated during pipe replacement planning, and it was determined that a large backlog exists of pipe that should be replaced in order to ensure stable water distribution.



In short, taking into account the condition of the DeWitt Springs Pipeline, and the wells and distribution network, it would be prudent to make long-overdue improvements to the water supply system before major failures occur.

Hydraulic Model Development

The GIS-based hydraulic model was developed in close collaboration with City staff from the GIS, Operations and Engineering Departments. The model was developed as an all-pipes model in H2OMAP™. This program allowed bringing aerial photos into the background, which made it quicker to identify areas where there were connectivity issues. Internal program tools helped quickly resolve many of these issues.

The model was calibrated based on SCADA operations data for the 12th of July 2004 – this day included both the 2004 maximum day and the highest peak hour demands for the two years 2004-2005.

Traditionally models have been calibrated by adjusting friction factors and other means to force the model to match documented operations data. However, with the all-pipes approach used for Logan, very little adjustment was actually necessary to obtain results that matched those recorded in the City SCADA system. For some smaller pressure zones pump curves had to be reduced, suggesting that impellers are worn or valves are pinched at the actual facility, but otherwise the model did not require adjustment.

The calibrated model was then subjected to four vulnerability analyses. Three scenarios analyzed the system with one different supply source out of service, and the fourth scenario analyzed a three-fire flow event. These analyses revealed the following:

- With one supply source out of service but all others operating, the water supply system can meet demands. Retiring the leaking Golf Course 1 and 2 tanks eliminated major water losses and provided additional storage.
- Pressures are so high in the lower part of the main zone that low pressures will almost never occur except locally with fire flows on mains smaller than 8" in diameter. Due to the stress of the physical infrastructure, and for public and City staff safety, reducing these remarkably high service pressures should take a very high priority.
- Some areas (including Castle Hills and Cliffside-Quail Bluffs) are connected to the water supply system by only one main. Should there be a failure on these mains, an entire small zone could be without water.



Together with City staff and considering State requirements, Black & Veatch prepared level of service goals for the future network. These are shown in Table ES.3.

Table ES.3 Minimum Level of Service Goals	
Category	Goal
Service Pressures	<ul style="list-style-type: none">• Minimum 20 psi for peak hour• Minimum 30 psi for maximum day with fire flow• Minimum 40 psi for maximum day• Maximum 100 psi where possible
Minimum Main Size	<ul style="list-style-type: none">• Minimum size 8-inch diameter to serve hydrants
Fire Flows	<ul style="list-style-type: none">• Fire 1000 gpm for residential areas• Fire 1500 gpm for buildings• Length of fire 2-4 hours for buildings
Storage	<ul style="list-style-type: none">• Equalization storage 400 gallons per ERC for indoor use• Equalization storage 2,848 gallons / irrigated acre
Dead End Mains	<ul style="list-style-type: none">• Avoid dead end mains• Provide fire hydrant or other flushing at dead ends
Corrosive Soils	<ul style="list-style-type: none">• Use plastic pipe

The calibrated model was then expanded to represent the 2025 network, which includes the large western annexation areas. New mains employed 12" minimum pipe diameter.

Based on evaluation of the existing system and taking the level of service goals into consideration, the future network would involve dividing the main zone into three pressure zones – an Upper Zone from the Golf Course tanks down to the existing line of PRV stations; a Central Zone from here west to about 10th West; and a Lower Zone from 10th West westwards. To divide the main zone into three zones would require the following changes to network configuration:

- Re-commission the existing PRV stations (and move the Center Street PRV farther east) to establish the Central Zone.
- Provide a second line of PRV stations along 10th West to establish the Lower Zone.
- Provide water to the Central Zone from the 6th East, Center Street, Crockett and Willow Park wells in addition to supply from the Upper Zone via the PRV stations.
- Provide flexibility to boost from the 6th East, Center Street and Crockett wells to the Upper Zone if the Golf Course tanks or DeWitt Springs Pipeline are out of service.
- Provide a dedicated main directly to the Golf Course tanks from the Center Street and Crockett wells.



- Provide additional storage in the Central Zone to ensure operations stability.
- Construct an additional well on the east side pumping directly to the Golf Course tanks via the dedicated main from the Center Street and Crockett wells.

Converting the main zone into the Upper, Middle and Lower Zones would almost certainly have to be staged in order to reduce annual capital costs:

1. In the first step, restore all six of the existing PRV stations to service to regulate pressures in the Central Zone. The Center Street, Crockett and 6th North wells would continue pumping to the Upper Zone and feed by gravity to the Central Zone. Willow Park well would be equipped with variable frequency drive and deliver to Central Zone pressures. The SCADA system would be upgraded as a prelude to better control of future system operations.
2. In the next step, construct transmission mains to allow delivery from the wells directly to the Central Zone. Reservoir capacity should also be constructed on the east and northeast sides of the Central Zone to provide operational stability.
3. In the third step, rehabilitate existing wells one at a time, and provide well pumps (with variable frequency drive) to supply to Central Zone pressures, and provide booster pumps able to supply to Upper Zone pressures.
4. Parallel to these steps, establish PRV stations on mains west of 10th West as new developments are approved, with cost sharing negotiated with the developers.

A series of other improvements were also foreseen to the future network, including two small PRV stations at low points in the Cliffside Zone, numerous new pipes to resolve hydraulic bottlenecks identified by modeling or by City staff, and implementing a general policy of upsizing small mains to meet fire flow requirements.

Tables ES.4 summarizes year 2025 pressure zones static pressures. Note that pressures in the former main zone are now reduced from 190 psi down to maximum 140 psi (Upper Zone), 100 psi (Central Zone) and 60 psi (Lower Zone). Similarly, in the Cliffside Zone, pressures are reduced from 186 psi to a maximum of 120 psi.



Table ES.4

Year 2025 Pressure Zones

Pressure Zone	Ground Elevation	Static Level (MSL)	Static Pressures (psi)	Controlling Reservoir
Upper Zone	4550-4840	4875	15-140	GC 1-4
Central Zone	4450-4550	4680	56-100	new tank(s)
Lower Zone	4430-4460	4570	48-60	new PRVs
Upper USU	4750-4846	5057	45-132	GC 5
Lower USU ¹	4650-4846	4930	36-122	PRVs
Castle Hills ²	4770-4940	5055	50-123	Castle Hills
Cliffside ³	4790-5016	5065	20-120	Cliffside
Quail Bluffs	4890-5102	5149	20-112	Quail Bluff

¹Upper USU and Lower USU may be operated together, but pressures would be extremely high.

²Castle Hills may be operated on the GC 5 level together with USU.

³PRV stations added to lower pressures in two small neighborhoods.

The future pipe network was then subjected to vulnerability analyses, much as had been performed for the existing system. However, vulnerability analyses for the future system considered failure of *two* supply sources simultaneously on a maximum day, which could easily occur in the event of a major earthquake.

In each case, the hydraulic analyses showed that the water supply system could handle a major event and still maintain a reasonable level of supply security. The only areas that were compromised in these runs were the USU/Castle Hills Zones, where failure of the Golf Course tanks or the DeWitt Springs Pipeline could result in loss of pressure.

System-wide fire flow analyses were also performed, and helped to locate key deficiencies in the network at locations such as the airport and the Island neighborhood, in addition to confirming the impact of fire flows on smaller mains.

The future network was then subjected to peak hour analysis, which proved that by incorporating some of the improvements identified in earlier analyses, the system would be able to perform quite well in 2025. Table ES.5 shows the results of this modeling.



Table ES.5 Year 2025 Peak Hour Model Results			
Pressure Zone	High Press. (psi)	Low Press. (psi)	Comment
Upper Zone	113	17	low pressure at high points near tanks, 24" main, and high point at closed USU valve
Central Zone	76	30	
Lower Zone	70	18	low pressure on 2" private line at composting facility; otherwise lowest pressure is 50 psi
USU Zones	116	30	
Castle Hills	119	49	
Cliffside	118	28	low pressure near tank
Quail Bluffs	118	22	low pressure near tank and at one high point

The results of the vulnerability, fire flow and peak demand analyses confirmed the ability of the proposed network configuration to handle future demands even in emergencies.

Following the year 2025 analyses, the year 2010 network was then analyzed with very similar results. It was assumed that in the year 2010 the main zone would be converted into three pressure zones, but that most expansion into the western annexation areas with the second line of PRV stations would not occur until after 2010.

GIS Review

GIS review confirmed that the GIS Division is taking appropriate steps to upgrade, develop and maintain a GIS database that will be of practical use for Engineering and Operations. The GIS database will be enhanced with a recently-purchased asset management system, CityWorks, but still needs to incorporate customer billing data.

The review also highlighted the need for implementing tools that Operations staff can use when in the field to document failures and repairs so that this information can be efficiently updated in the database maintained by the GIS Division.

The review evaluated options for GIS and hydraulic model integration. Black & Veatch recommends that the City continue with H2OMAP modeling (since the model is already developed in H2OMAP), or convert to InfoWater since the model can be imported directly into InfoWater, which has seamless GIS integration.



This section also included the task Corrosion Attribute Assignment. The GIS Division used information provided by Black & Veatch to incorporate the USDA NRCS soils database into the GIS. The GIS Division further developed a routine to assign soils corrosivity potential to the pipelines. It was determined that almost the entire network is located in soils with high corrosion potential for cast iron and ductile iron pipe. The NRCS data was used to develop a soil corrosion potential rating that was then used in pipe replacement planning.

Pipe Replacement Plan

In this task, Black & Veatch developed pipe replacement prioritization categories, scores and weights together with City staff, assigned these scores and weights to all network piping, developed water main survival curves with unit replacement costs, and integrated prioritization with replacement costs to develop budget options.

This is a valuable planning tool to help identify cost-effective packages for pipe replacement. Eventually all piping in any network will need replacement over 50-100 years, but it is most cost-effective to replace pipe that is or will soon fail or cannot meet other objectives. Pipe replacement planning helps by weighing cost and non-cost aspects together.

The primary output is a defensible long-term maintenance budget for pipe replacement.

Prioritization categories selected were pipe age, leaks/mile, service pressure, soil corrosion potential, criticality (key supply mains or service to key institutions like hospitals and schools that can function as emergency shelters), coordination with other projects (such as road or sewer improvements), and supervisor input.

Together with City staff it was determined that it would be most realistic to weight pipe age and leaks/mile more heavily than other factors. This resulted in realistic prioritization packages that were easily identifiable in GIS as long stretches of mains primarily in the older central business district, but also in some newer areas.



City input (coordination with other projects and supervisor input) further helped develop clear packages for pipe replacement. An evaluation of City input showed that City staff had been selective in utilizing these prioritization categories, and hydraulic modeling confirmed City priorities.

The next step was to determine what water main survival curves would best predict the anticipated lifetime of pipe over time. Some pipe may begin to fail after only 10 years, while other pipe may have a useful lifetime exceeding 100 years. This has been the case with the City of Logan network.

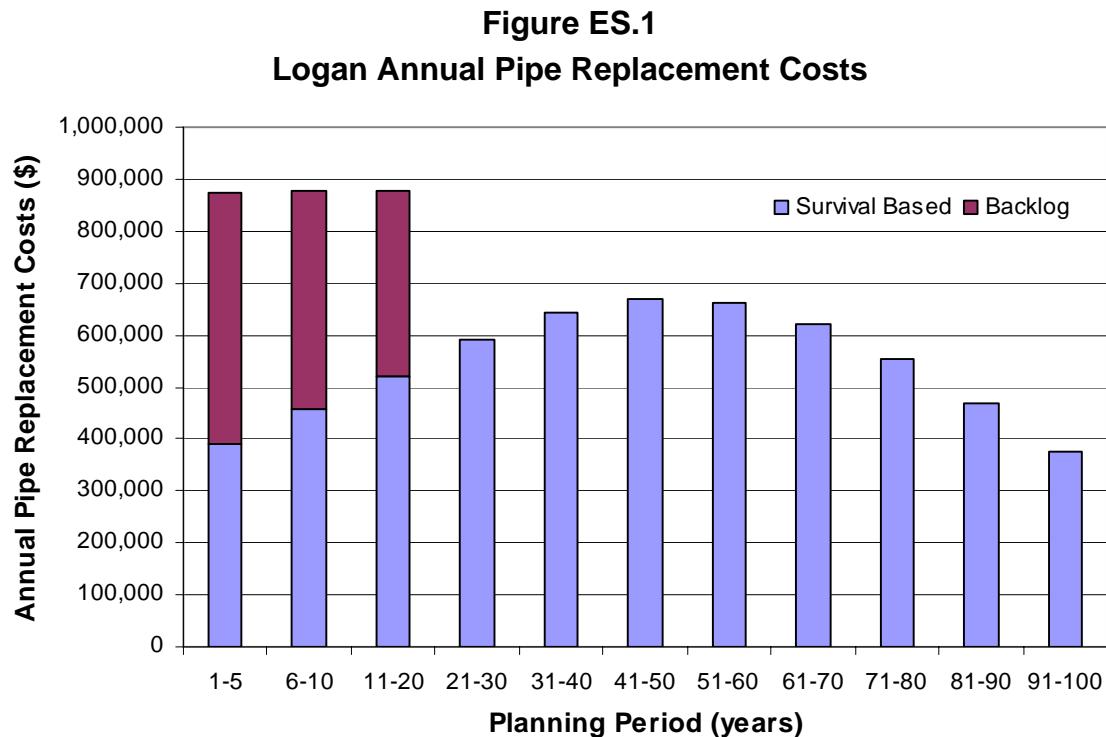
Considering the soil corrosivity potential, high pressure and history of pipe breaks, it was determined that the Bellevue High Deterioration survival curve most closely approximated City of Logan experience. This curve accounts for both long pipe lifetime and early failure of some pipe, just like in Logan.

The selected water main survival curve can then be applied to the GIS files showing age of every pipe in the City network in order to obtain an idea of how much network piping will need to be replaced in any given year. With this information the City can develop approximate annual pipe replacement budgets. These budgets can be adjusted over time as the network ages and as new pipe is installed.

The resulting Pipe Replacement Plan, expressed in terms of annual replacement costs, is shown in Figure ES.1. A budget for addressing pipe replacement backlog was added per City request to adequately represent the full replacement costs which can be anticipated over the next century.

The Pipe Replacement Plan provides a powerful indication of when investments in pipe replacement will “come due” and how much this will cost on an annual basis over the planning period.

The City can also use this data to track the backlog of replacement needs as they grow or decline, depending on whether the funds made available for network rehabilitation exceed the needed level of investment to maintain status quo.



Regulatory Review

The Regulatory Review looked into current and anticipated regulatory requirements, and compared relevant requirements against historic water quality data for each Logan supply source. The review concluded that:

- The Consumer Confidence Report needs to be made available on the City website.
- The City should sample for radon to document ahead of the upcoming Radon Rule.
- The City should verify compliance with disinfection requirements of the Ground Water Rule by sampling for free chlorine at the first users along the DeWitt Springs Pipeline.
- The City should document Crockett well compliance with the Arsenic Rule.

Capital Improvements Plan

The Capital Improvements Plan identified six main categories for water supply system investments, with investments divided over five-year increments:

- Conversion of the main zone to three pressure zones.
- Provision of redundancy to reduce water supply system vulnerability.
- Improvements to resolve local delivery problems.
- Expansion of the service area westwards.



- Additional storage as required by the Utah Division of Drinking Water.
- Implementation of the Pipe Replacement Plan.

Table ES.6 shows the resulting Capital Improvements Plan summary of costs to resolve all of these issues by the proposed year 2025 planning horizon.

Package	Package Costs in \$million				
	2007-2010	2011-2015	2016-2020	2021-2025	Total Cost
1. Convert Main Zone	\$1.60	\$5.84	-	-	\$7.44
2. Provide Redundancy	\$8.29	\$3.29	\$8.58	-	\$20.16
3. Resolve Delivery Problems	\$3.24	\$6.05	\$1.69	-	\$10.97
4. Service Area Expansion	\$7.32	\$9.15	\$9.15	\$9.15	\$34.76
5. Additional Storage	-	-	-	\$13.16	\$13.16
6. Pipe Replacement Plan	\$3.50	\$4.38	\$4.38	\$3.51	\$15.77
Total	\$23.94	\$28.70	\$23.79	\$25.81	\$102.25
Annual Cost each period	\$5.99	\$5.74	\$4.76	\$5.16	

The Logan Water Master Plan shows needed investments, and attempts to phase these in a logical manner in order to achieve City objectives within the year 2025 planning period.

The City of Logan must decide the level of commitment to make in any given year or planning period. This is understandably subject to change based on trends in revenues, funding opportunities and political will.

City Operations staff are already making important contributions to rehabilitating the network under existing operations budgets. But it is clear that significant capital investments will be needed to prevent disruption of the water supply system and head off increasing pipe failures, expand the network to supply new developments, and comply with State of Utah requirements for public water systems.



1.0 Introduction

1.1 Scope of Services

The City of Logan requested Black & Veatch in August 2005 to prepare the 2005 Water Master Plan for the Existing Culinary Water System. The primary components of the project include:

- An update of the existing EPANET distribution system model including evaluation of historical water use and water use rates, evaluation of future water demand projections, and allocation of water demands to the hydraulic model.
- Model validation and evaluation of the existing system with respect to efficient operation, safety, and level of service goals.
- Development of proposed improvements for years 2005 and 2025 based on projected water demands.
- Vulnerability analyses to identify improvements that will provide a higher degree of system reliability and operational flexibility now and in the future.
- A review of the existing City GIS as it relates to water supply and distribution and recommendations for improvements and additions. Review of tools available to integrate the City GIS with the hydraulic model.
- Development of a pipeline replacement plan and estimate of annual expenditures required to maintain an acceptable level of service for City customers.
- Safe Drinking Water Act regulatory review to incorporate the requirements of pending and future drinking water regulations into planned improvements as required.
- Update of the City Water Conservation Plan in compliance with the State of Utah Division of Drinking Water requirements.
- Compilation of project deliverables into the City of Logan Water Master Plan.

1.2 Change in Scope to GIS-Based Approach

Considering the stated City objective of more closely integrating the hydraulic model with the information available in the City GIS, and the very schematic nature of the EPANET model, Black & Veatch proposed converting to a GIS-based hydraulic model where GIS files could be imported into H2OMAP.

Black & Veatch did not request additional funds for this change in scope, though it would require more work. The City of Logan approved this in August 2005, and requested that the model be an all-pipes model. The resulting model has more than 5000 nodes.



The change in scope to a GIS-based approach required a much longer schedule for completing the Master Plan. This is reflected in major City deliverables listed below, which were prerequisite for expediting the master planning activities.

- The City commenced updating and correcting water distribution network information in the GIS. The first batch of GIS files, as well as City-approved population projections, was delivered to Black & Veatch at the end of November 2005. The City provided additional GIS files at the end of February 2006.
- In March 2006 the City expanded the scope of the master plan to include future western annexation areas, in effect doubling the future service area. The City also developed completely new population projections now including these western annexation areas.
- In April 2006 the City delivered pipe age information to be used in the GIS-based pipeline replacement planning, and final SCADA production data, to be used in calibrating the hydraulic model.
- In November 2006 the City provided modified population density and projections requiring model modifications.
- City review of the draft Water Master Plan report in early 2007.

It should be noted that Black & Veatch submitted the Water Conservation Plan in December 2005. The City of Logan reviewed and submitted it to the Utah Division of Water Resources, which approved the plan. However, the Water Conservation Plan was prepared prior to the change in scope to include the western annexation areas. Therefore the plan does not reflect updated population and water demand projections.

Updates to the Water Conservation Plan are scheduled for fiscal year 2010. The plan submitted to the State of Utah is provided in the Appendix.

1.3 Abbreviations

AARC	Annual Average Rate of Change
ADF	Average Day Flow
AMCL	Alternate Maximum Contaminant Level
AWWA	American Water Works Association
B&V	Black & Veatch
cfs	cubic feet per second
CIP	Capital Improvement Plan



CT	Contact-Time
DBP	Disinfection By-Products
D/DBPR	Disinfection/Disinfection By-Products Rule
DDW	Utah Division of Drinking Water
DWR	Utah Division of Water Rights
DWRe	Utah Division of Water Resources
EPANET	EPA hydraulic modeling program
EPS	Extended Period Simulation
ERC	Equivalent Residential Connection
ESWTR	Enhanced Surface Water Treatment Rule
GC	Golf Course
gpcd	gallons per capita day
GIS	Geographical Information System
GOPB	Utah Government Office of Planning and Budget
gpm	gallons per minute
GPS	Global Positioning System
GWR	Ground Water Rule
HAA5	Halo-Acetic Acids (five)
HGL	Hydraulic Grade Line
hp	horsepower
H2OMAP	MWH Soft hydraulic modeling system
ICI	Industrial-Commercial-Institutional
IDSE	Initial Distribution System Evaluation
MCL	Maximum Contaminant Level
MDF	Maximum Day Flow
µg/L	microgram per liter
MG	Million Gallons
mgd	million gallons per day
mg/L	milligram per liter
mi	mile
MM	Maximum Month demand
MMM	Multi-Media Mitigation
ND	Minimum Day demand
NPSH	Net Positive Suction Head
p.a.	per annum (per year)
pCi/L	pico-Curies per liter



PHF	Peak Hour Flow
PRV	Pressure Reducing Valve
psi	pounds per square inch
SCADA	Supervisory Control And Data Acquisition
SDWA	Safe Drinking Water Act
SMCL	Secondary Maximum Contaminant Level
SOC	Synthetic Organic Chemical
TAZ	Traffic Area Zone (US Census Bureau population district)
THM	Trihalomethanes
USDA	United States Department of Agriculture
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
USU	Utah State University
UV	Ultra-Violet irradiation
VFD	Variable Frequency Drive
VOC	Volatile Organic Chemical

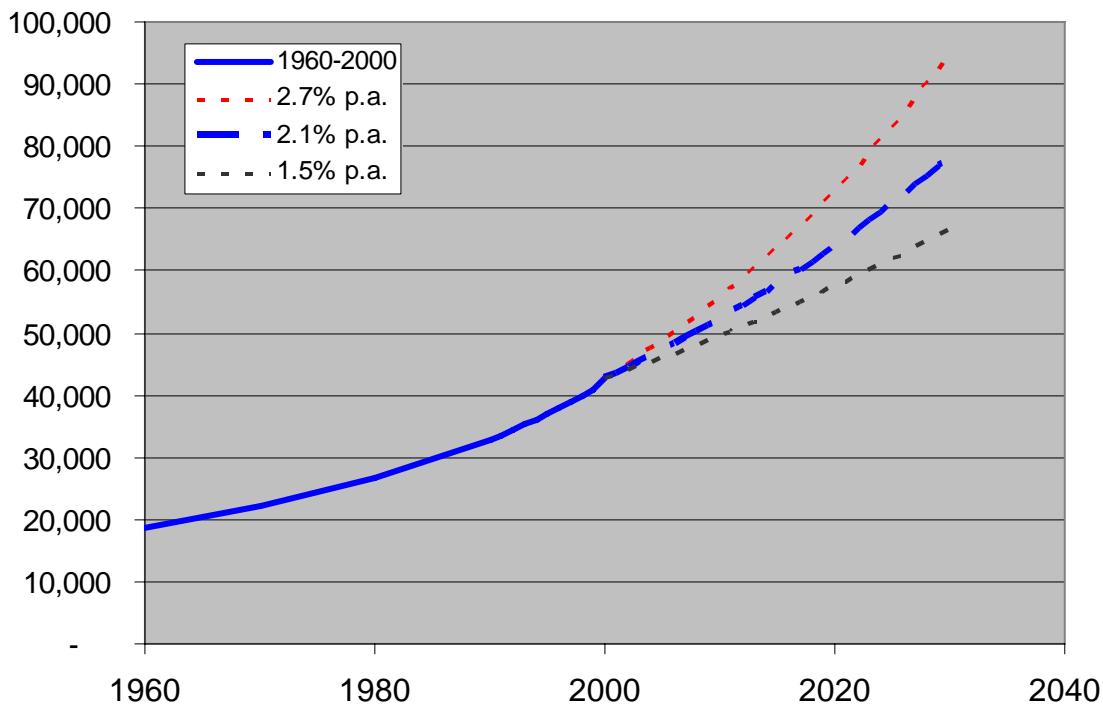


2.0 Demographics and Water Use Projections

2.1 Historical and Future Demographics

City of Logan growth rates have consistently exceeded Cache County growth rates and GOPB projected growth. Growth rates over the period 1960-2000 averaged 2.1% AARC; and 2.7% AARC over the last ten years of that period. The City is of the opinion that future growth rates will most likely be about 2.1% AARC, and in worst case 2.7% AARC. Figure 2.1 shows the past trends and projected population in the current service area. This information was provided in the Water Conservation Plan.

Figure 2.1
Logan Historic and Projected Population, 1960-2030



Year 2025 population was estimated at 70,905 persons, and employment was estimated at 52,356 persons. The large employment component is a reflection of Logan's role as the commercial and institutional hub of Cache County.

Since production of the Water Conservation Plan, the City instructed Black & Veatch to include western annexation areas into the scope of master planning.



The inclusion of the western annexation areas increased the estimated population by an additional 5% in the year 2025 to 74,441 persons. Employment increased by 27% to 66,506, reflecting planned commercial-industrial developments on the west side. This information is shown in Table 2.1, and illustrated in Figure 2.2.

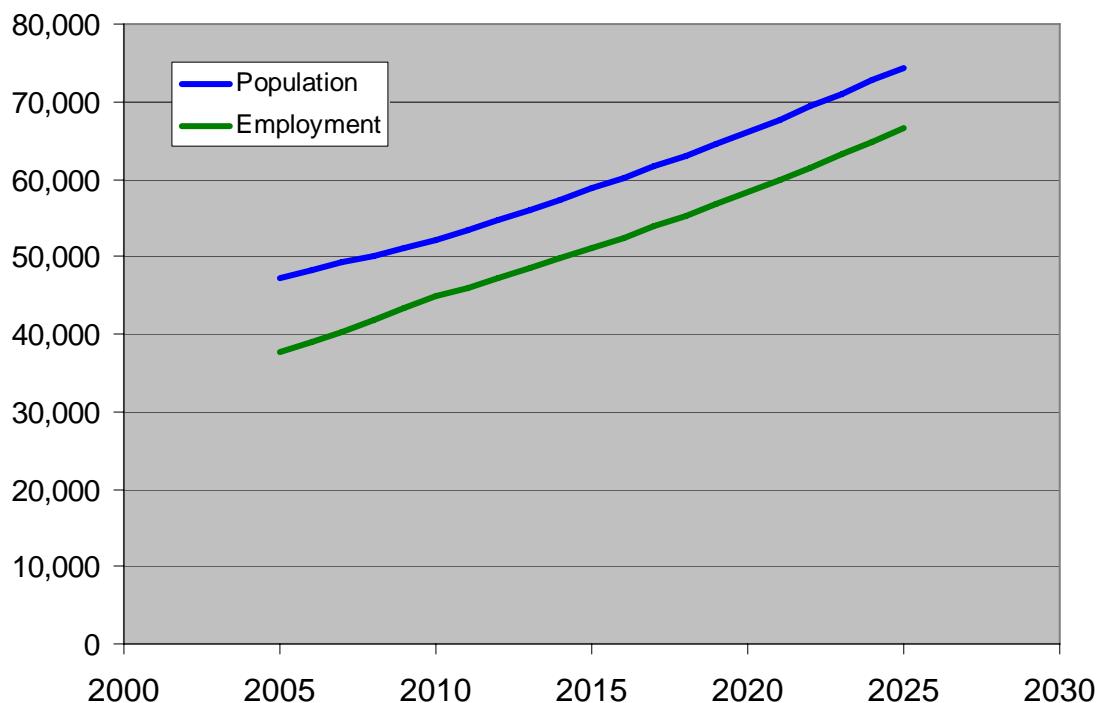
Table 2.1		
Logan Projected Population and Employment, 2005-2025¹		
Year	Population²	Employment³
2005	47,235	37,600
2010	52,226	44,847
2015	58,775	51,142
2020	66,146	58,320
2025	74,441	66,506

¹Includes western annexation areas.

²Projected annual growth averages to an AARC of 2.0% to 2010, and 2.4% from 2010 to 2025.

³Projected annual growth averages to an AARC of 3.6% to 2010, and 2.7% from 2010 to 2025.

Figure 2.2
Logan Projected Population and Employment, 2005-2025





Population and employment data from the City was then adapted to the planning years (2010 and 2025) and distributed to each TAZ in GIS before allocating water demands.

2.2 Unit Water Consumption

Water demands can be represented in terms of per capita consumption in order to relate to State water conservation goals and better identify trends in consumption by category.

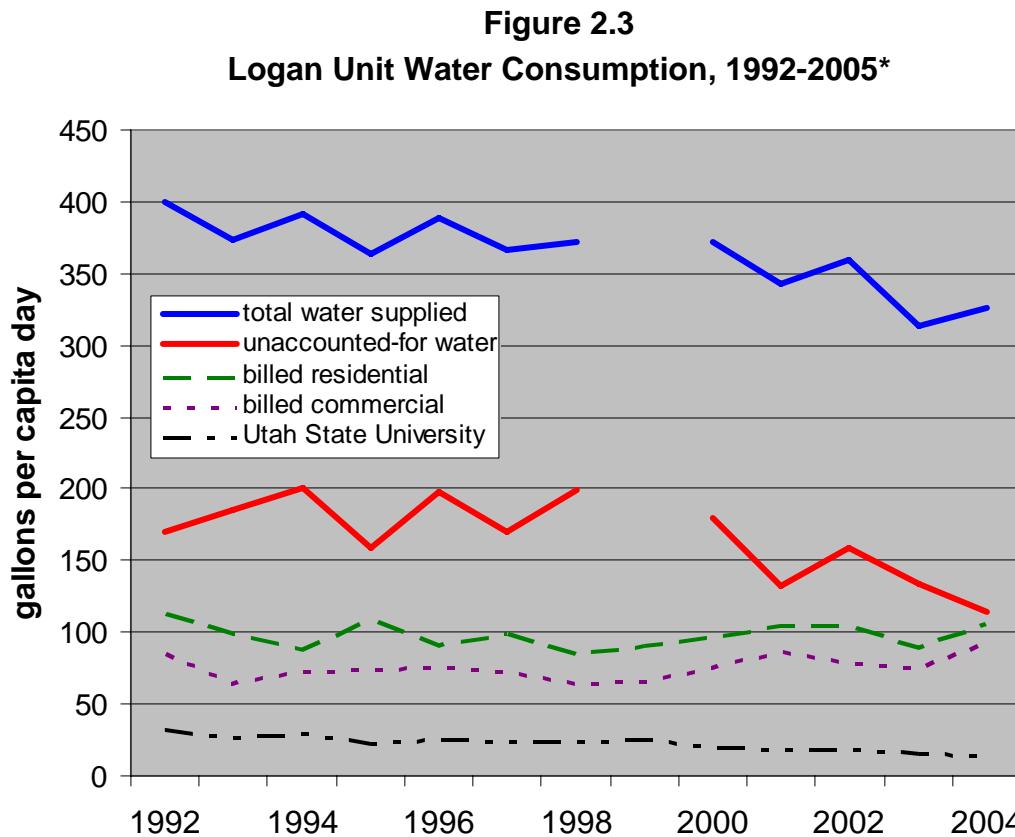
The City of Logan historically lists two categories for billed water consumption: residential consumption, and commercial consumption. Residential consumption includes residences, but also apartments with master meters; commercial consumption includes industry and institutions, City offices and facilities, and parks irrigation from culinary supply. The City also keeps accounts for water sold to USU.

The difference between billed water consumption and total water supplied to the system is referred to as unaccounted-for water. Unaccounted-for water includes water consumed but not billed, fire flows, other unmetered municipal uses, and water lost to leakage in the water supply system. Unbilled water consumption can owe to under-registering customer meters, and poor water accounting and billing practices.

Figure 2.3 provides a unit water consumption comparison between billed (residential, commercial, USU) and unaccounted-for water use. The figure also shows the total amount of water supplied to the system for comparison. The sum of the residential, commercial, USU and unaccounted-for water use is equal to the total water supplied. The gap in data for 1999 owes to unreliable information for total water supplied that year.

Residential billed consumption averaged about 100 gpcd in a flat trend over the 12-year period. This compares favorably with year 2003 residential consumption of 140 gpcd in Salt Lake City, and with the *year 2025 goal* for residential consumption of 130 gpcd for West Jordan. It also means that water conservation programs targeting residential consumption in Logan will have only limited impact on reducing overall water demands.

Commercial billed consumption includes industries and institutions. However, USU has been broken out as a separate category for purposes of discussion. Commercial billed consumption averaged about 70 gpcd through the 1990s, but has risen since 2000 in response to expansion in the commercial sector.



*Year 1999 data for total water supplied was not available.

USU billed consumption by 2004 had fallen to 13 gpcd, even though USU student enrollment expanded to 23,908 students in 2004. USU has its own well, as well as canal water for irrigation, so it is not clear if this trend owes to a drop in USU consumption, but at least it has reduced demands upon the municipal culinary water supply system.

Commercial billed consumption (incl. USU), corresponds to 50% of total billed, underlining Logan's role as the economic hub of a metropolitan area serving outlying communities and high water demand industries. It can be expected that the commercial sector will continue to contribute a large proportion of total consumption in the future.

Unaccounted-for water comprised the single largest component of water consumption, averaging 42% of water supplied in 2000-2004. It represents the most obvious place for a water conservation program to begin. Unaccounted-for water averaged over 180 gpcd during the 1990s, but since then has been reduced to 114 gpcd in 2004.



Total water supplied to Logan – the sum of residential and commercial consumption, and unaccounted-for water – averaged 380 gpcd during the 1990s, but fell to 325 gpcd in 2004, bringing the City closer to meeting the DWRe statewide goal of 240 gpcd by 2050.

2.3 Historical Water Use Rates

The City of Logan water supply system must be able to deliver water to customers at rates which fluctuate widely. Yearly, monthly, daily and hourly variations are to be expected, and water use typically follows seasonal and diurnal patterns.

Average day use is the total annual water use divided by the number of days in the year. The average day rate is used primarily as a basis for estimating maximum day and peak hour demands. The average day rate can also be used to estimate future supply requirements, revenues and operating costs.

Maximum day use is the maximum water used on any one day of the year. The maximum day rate is used to size water treatment facilities. Pumping or gravity source facilities must be capable of providing this quantity of water as a minimum.

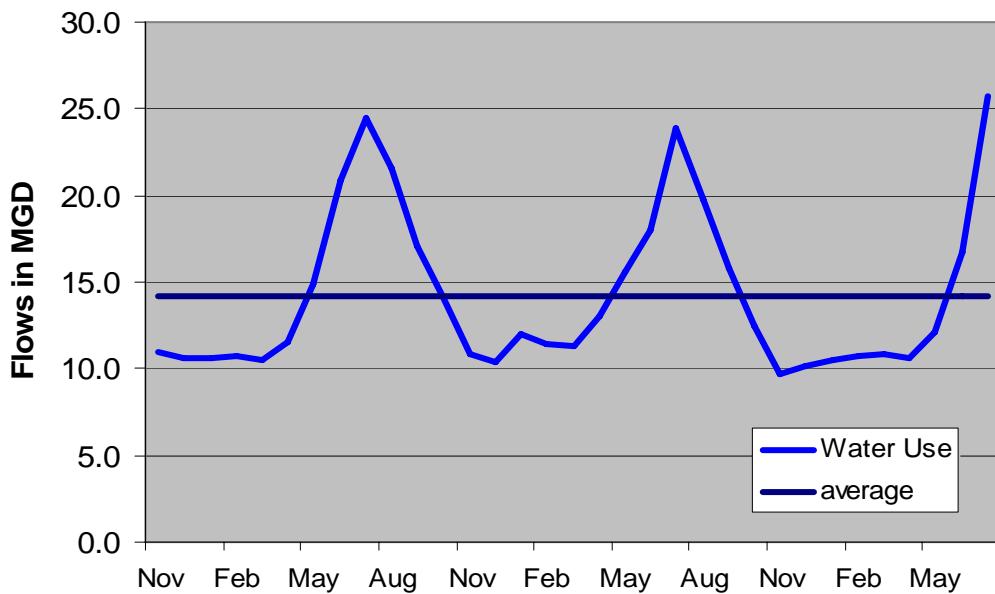
Peak hour use is the maximum water used in any one hour of the year. Since minimum system pressures are usually experienced during peak hour, the sizing and locating of distribution facilities are generally determined on the basis of this condition. The difference between maximum day and peak hour water requirements is most efficiently met by strategically located system storage. This minimizes the required capacity of pump stations, transmission mains and treatment facilities; and permits a more uniform and economic operation of the water supply system.

Maximum month water use establishes sustained, long-term maximum water use rates in the system. Minimum day water use can be used to establish the maximum water age in the system and the ability of existing and future infrastructure to maintain acceptable water quality.

Figure 2.4 shows seasonal variations in water use during the period 2002-2005, where summer monthly water use was 1.7-1.8 times the annual average demand. It also indicates that overall water consumption did not change significantly during this period.



Figure 2.4
Logan Monthly Water Use, November 2002-July 2005



The City recently installed a SCADA system, which makes it possible to track water supplied to the network on a daily and even real-time basis. Figure 2.5 documents daily water supplied from DeWitt Spring and total water supplied for the same period.

Maximum day demands are 1.8-2.0 times the annual average demand. Note also that flow from the DeWitt Spring is unable to meet daily demands for about five months of the year, due to hydraulic restrictions and water rights limitations. The City relies heavily on pumping from the groundwater wells during these periods.



Figure 2.5
Logan Daily Water Supply, November 2002-July 2005

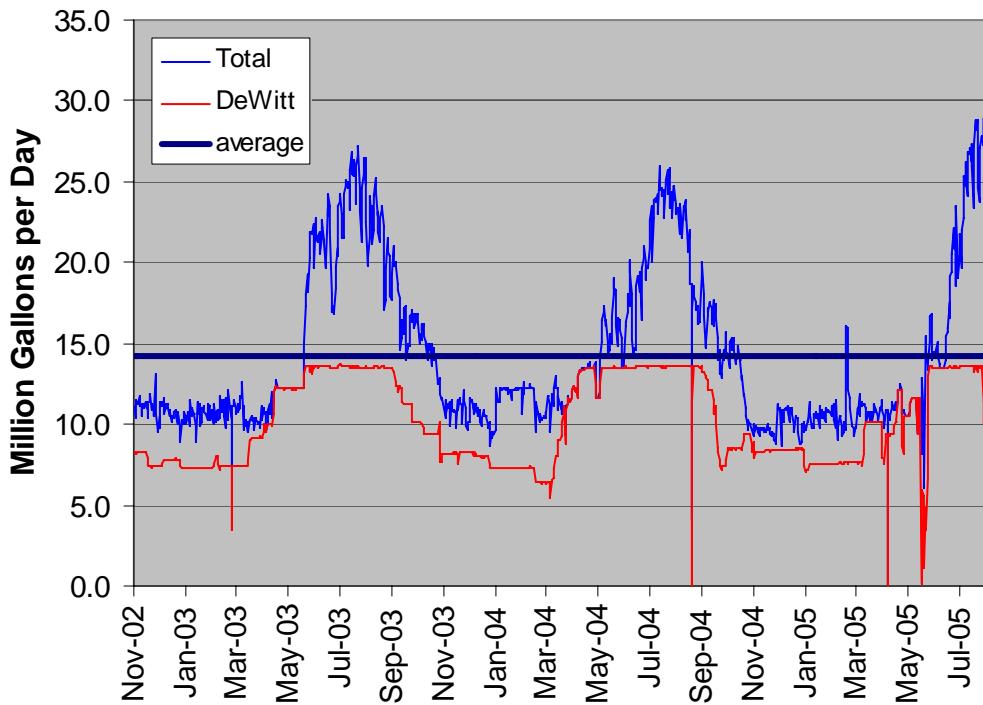


Figure 2.6 documents diurnal demands over two 36-hour periods centered around the annual maximum day demands on the 12th of July 2004, and the 29th of July 2005. These are documented in quarter-hour segments, and show peak hour demands of about 35 mgd (2004) and 29 mgd (2005). Peak demands exceeded the average annual demand by a factor of 2.2 to 2.5 during these two periods.

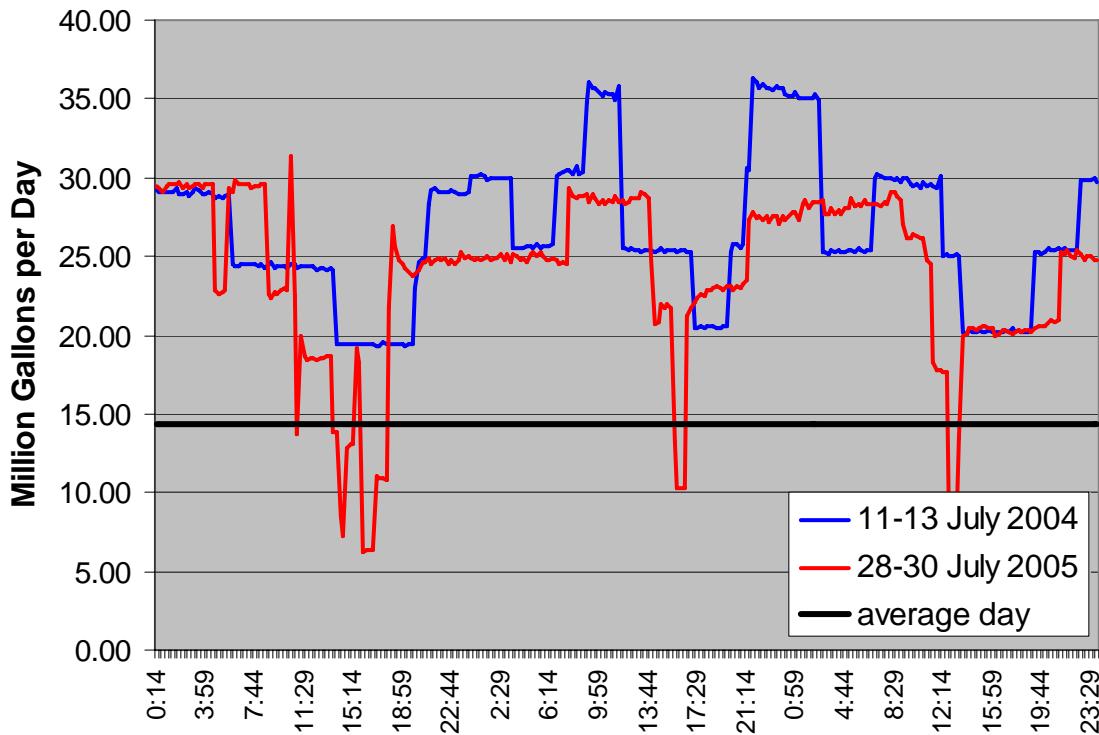
The peak hour for 2004 can be approximated at 36.0 mgd – quarter-hour production data was not available for the Willow Park well where data was averaged over the day. The peak hour demand in 2005 actually occurred on the 14th of July, at 30.4 mgd.

Average daily water supply during this period was 14.3 mgd.

As the City compiles a better and more accurate historic production record, the peaking factors may be refined and trends may become more discernible.



Figure 2.6
Logan Diurnal Demands on Maximum Demand Days, 2004 and 2005



Tables 2.2 provides a summary of water use rates and peaking factors based on information available for the period January 2003-July 2005.

Table 2.2			
Logan Water Use and Demand Factors, January 2003-July 2005			
Category	2003	2004	2005
Annual Average Day (AD), mgd	14.83	14.49	14.02
Maximum Day (MD), mgd	27.20	25.99	28.90
Peak Hour (PH), mgd	NA	35.96	30.39
Minimum Hour (ND), mgd	7.46	8.68	6.07
Maximum Month (MM), mgd	24.45	23.87	25.73
Peak Hour/Average Day factor		2.48	2.17
Maximum Day/Average Day factor	1.83	1.79	2.06
Peak Hour/Maximum Day factor		1.38	1.05
Maximum Day/Maximum Month factor	1.11	1.08	1.12
Minimum Hour/Average Day factor	0.50	0.60	0.43



When a longer, more accurate historical record has been developed, it will be possible to create frequency-distribution plots to provide more precise estimates of peaking factors.

For hydraulic modeling purposes for the future network, a MD/AD peaking factor of 1.8 was employed. For peak hour flows, a PH/AD factor of 2.4 was employed. Peaking factors may decline in the future due to population growth, water conservation, and better management of secondary watering, but this has not been assumed in modeling.

2.4 Water Demand Projections

Population and employment projections, unit water consumption by category, and unaccounted-for water were used to estimate average day water demands for the years 2010 and 2025. Projections for year 2010 and 2025 unit water consumption assumed that:

- Residential consumption stays constant at 100 gpcd.
- Commercial consumption stays constant at 90 gpcd.
- USU consumption stays constant at 20 gpcd.
- Unaccounted-for water, assumed at 125 gpcd, is reduced by 3.3 mgd due to the City taking the leaking Golf Course tanks out of service, and reduced by another 1.0 mgd by planned replacement of the lower section of the DeWitt Springs pipeline.

The population and employment projections were tied to TAZ, unit consumption was applied and the resulting demands from each TAZ were added to the individual nodes. Table 2.3 shows the resulting demand projections compared against 2004 data.

Table 2.3
Logan Water Demand Projections, mgd

Category	2003-2005	2010	2025
Annual Average Day			
Residential	4.50	5.22	7.44
Industrial-Commercial-Institutional	4.48	4.70	6.70
Unaccounted-for-Water	5.66	1.70	2.16
Total	14.63 ¹	11.62	16.31
Maximum Day	26.60 ¹	20.85	29.29
Peak Hour	33.18 ²	27.77	39.02

¹Average of 2003-2004.

²Average of 2004-2005.



The noteworthy difference between existing and future demands is the reduction in unaccounted-for water at the Golf Course tanks and DeWitt Springs pipeline, which in essence buys the City of Logan additional supply capacity.

The demand projections do not take into account savings in unaccounted-for-water that will result from continued leakage detection and repair, and lower leakage rates due to reducing pressures in the network; nor reduced unit consumption for residential and commercial demand if stricter water conservation measures are implemented.



3.0 Existing Distribution System Facilities

3.1 Water Distribution System Supply Sources

The City of Logan is supplied from the DeWitt Springs in Logan Canyon, and from four wells at Center Street, Crockett, 6th East 7th North, and Willow Park. The 10th North well supplies irrigation exchange water allowing increased use of the DeWitt Spring source.

Table 3.1 shows the historical production capacity from the five culinary sources during the period November 2003 to July 2005. The wells are not operated continually – hence the low annual average day production values are not representative.

Table 3.1		
Logan Historical Production by Source, November 2003-July 2005		
Source	Average, mgd	Peak day, mgd
DeWitt Springs	10.19	13.69
Center Street Well	0.69	5.27
Crockett Well	1.06	6.22
6 th East 7 th North Well	1.78	5.12
Willow Park Well	0.55	5.29
All sources	14.26	28.90

3.2 Pressure Zones

Ground elevations within the City of Logan service area range from just above 4,400 feet (USGS datum) on the west side of the city to about 5,150 on the east bench. The service area is comprised primarily of one large pressure zone, with several smaller secondary pressure zones served via booster stations from the primary pressure zone.

Table 3.2 shows the pressure zones and their static service levels (in MSL, Mean Sea Level) and static pressures. The main zone has extremely high dynamic pressures that rise from 120-130 psi near 3rd East, to above 220 psi on the west side. This high pressure represents a serious safety hazard to utility repair staff, a hazard to the public and property during dramatic pipe failures, a continual strain on the pipe network that shortens overall pipe life, a source of high leakage rates, and an unnecessary waste of energy for the well pumps to meet these high pressures.



Table 3.2 Pressure Zones				
Pressure Zone	Ground Elevation (MSL)	Static Level (MSL)	Static Pressures (psi)	Controlling Reservoirs
Main Zone	4430-4840	4875	15-190	GC 3,4,6,7,8,9
Upper USU	4750-4846	5057	45-132	GC 5
Lower USU	4650-4846	4930	36-122	PRVs
Castle Hills	4770-4940	5055	50-123	Castle Hills
Cliffside	4635-5016	5065	20-186	Cliffside
Quail Bluffs	4890-5102	5149	20-112	Quail Bluffs

It should also be noted that Castle Hills and the Upper USU zone are operated as one pressure zone controlled by GC 5. A PRV station has been installed to lower pressure to Castle Hills, but this is not currently operated.

The Lower USU zone is separated by USU PRV stations from the Upper USU zone, and can be operated at lower pressures or as part of the Upper USU zone, in which case pressures in the Lower USU zone would be even higher than shown above.

Utah DDW prefers to see maximum service pressures in the range of 60-80, but the network is currently not configured this way. The low static pressures in the table above are usually nodes nearest the controlling reservoir, and do not imply that large areas are subject to low pressures. The extremely high static pressures in the Cliffside zone are located along a few streets located down in a canyon wash.

It should be mentioned that not only are there three operating PRV stations associated with the USU service areas, but there are an additional six PRV stations along a north-south line from Center Street and 2nd East north to about 14th North 330 East. These stations were constructed some 10 years previously, but when they were commissioned, complaints from industry and other customers on the west side due to lower delivery pressures convinced the City to cease operating these PRV stations. This issue will still need to be addressed should the City decide to lower service pressures in the future.



Main Pressure Zone

The main zone includes about three-fourths of City water demands. The main zone is lower than the USU and Cliffside areas, and includes the old part of town and all the areas to the west.

The four wells are needed to maintain pressure in the *upper* reaches of this main zone during peak demand periods, with the result that dynamic pressures can reach 220 psi in the *lowest* areas of the main zone, as confirmed by both fire hydrant test data and modeling of the existing system.

The main zone is fed in part from the upper areas by two 24" transmission mains from the GC tanks. One of these mains drops out at 8th East, while the other continues past the 6th East well all the way to 1st West, where it ties into small-diameter pipes. However, it also ties into a matrix of laterals that help to disperse flows throughout the main zone.

The rest of supply comes from the four wells. The Crockett, Center Street and 6th East wells are connected by 16" transmission mains; the one at 6th East ties into the remaining 24" transmission mains. The Willow Park well only has 10" mains tying into the system.

On the west side of the existing main zone there are shorter, often isolated, segments of 18" transmission mains. These have been installed piece-meal as the need arose for new or replaced pipe, keeping the 1997 Water Master Plan in mind, which called for major transmission capacity on the west side, particularly in a north-south direction.

Castle Hills and USU Pressure Zones

Both Castle Hills and the USU zones are fed from the GC 5 tank by a single transmission main, so there is no redundancy in supply to this area. The lower USU zone is fed through PRV stations, while the Castle Hills zone is fed by the Castle Hills booster station. Castle Hills has its own tank to maintain pressure. The USU zones also have their own well on USU campus.

Cliffside and Quail Bluffs Pressure Zones

The Cliffside zone is fed via the Cliffside II booster station from a single main off the primary zone, with no redundancy for emergencies. The Cliffside I booster station is adjacent to Cliffside II but is not operational and the pumps are removed. The Cliffside zone has a tank to maintain stable pressure.



The Quail Bluffs booster station pumps water from the Cliffside zone into the Quail Bluffs zone. Thus any failure in supply to Cliffside would also leave Quail Bluffs with no alternative supply. The Quail Bluffs zone has its own tank to maintain stable pressure.

3.3 Physical Infrastructure

The City of Logan water supply system consists of these key features:

- DeWitt Springs and transmission main through Logan Canyon to the GC tanks.
- Center Street, Crockett, 6th East and Willow Park wells.
- Tanks at the Golf Course; and at Castle Hills, Cliffside and Quail Bluffs.
- GC5, Castle Hills, Cliffside II and Quail Bluff booster stations.
- Three PRV stations in the USU area, and six PRV stations in the main pressure zone.
- Transmission mains and distribution pipelines.

DeWitt Springs and Pipeline

The DeWitt Springs on an annual basis provide most water to the City. This source is high-quality water, as evaluated in the regulatory review, and is conveyed by gravity to the municipal network. DeWitt Springs serves as the base flow year-round.

The springs are covered and protected, but in 2006 the adjacent river backed up and nearly flooded the springs, showing that surface contamination is possible. Flow from the springs is conveyed to an effluent weir and thence into the transmission main to the City. A gas chlorine injection system at the springs is used to maintain residual in the water.

The 5.5 mile DeWitt Springs pipeline consists of 36", 30" and 24" segments of concrete pipeline, which according to the CH2M Hill study is in good condition. Farther down Logan canyon, the pipeline is 24" and 20" steel. The last segment experiences both the highest pressures and according to the CH2M Hill study is in the worst condition, with leakage estimated at 1.0 mgd in this section.

Improvements to the DeWitt pipeline have been the focus of a separate study by CH2M Hill and therefore this pipeline is not further addressed in this report, except that the cost for DeWitt pipeline improvements is incorporated in the CIP in section 8.



Groundwater Wells

The potable water supply system is fed by four wells, Center Street, Crockett, 6th East, and Willow Park. Table 3.1 shows the historic maximum production from each of these wells. Water quality is high as reported in Section 7: Regulatory Review.

With the exception of Willow Park, these wells are 30 or more years old, and very little reliable information can be obtained concerning pump curves and other information. Also, the Center Street and 6th East wells consist of a submersible well pump coupled in-line with an above-ground centrifugal booster pump. All the wells except for Crockett have on-site backup power generators.

None of the wells have variable-frequency drives (VFDs) to stabilize production in response to changes in pressure at designated points in the network, but the pumps are connected to the City SCADA system for start-stop and data logging purposes.

Table 3.3 provides summary information about the wells, where some information was incomplete and had to be interpolated or assumed.

Table 3.3
Existing Potable Well Production Facilities

Well	Pump	hp	Capacity (mgd)	Head (feet)
Center Street	Well pump	200	4.3	139
	Booster pump	200	6.5	288
Crockett	Well pump	700	6.2	500
6 th East	Well pump	250	4.7	231
	Booster pump	250	5.0	250
Willow Park	Well pump	500	4.7	437

With the exception of the Willow Park well, it can be said that these wells are near or past their useful economic lifetime. The condition of the well screens is unknown.

Storage Reservoirs

The water distribution system includes ground storage tanks above the Golf Course, as well as smaller ground storage tanks in the Castle Hills, Cliffside and Quail Bluffs zones. Table 3.4 provides information on each of these storage tanks.



Table 3.4

Existing Storage Facilities

Name	Volume (MG)	Sidewater Depth (ft)	Overflow Elev. (MSL)	Pressure Zone
Golf Course 3	1.1	15.5	4,869.28	Main zone
Golf Course 4	1.1	15.5	4,869.29	Main zone
New GC6 ¹	1.3	21.0	4,875.0	Main zone
New GC7 ¹	1.5	21.0	4,875.0	Main zone
New GC8 ²	1.5	21.0	4,875.0	Main zone
New GC9 ²	1.4	21.0	4,875.0	Main zone
Golf Course 5	2.0	17.75	5,056.67	USU
Castle Hills	0.5	15.35	5,065.68	Castle Hills
Cliffside	1.1	14.85	5,065.12	Cliffside
Quail Bluffs	0.2	11.42	5,149.02	Quail Bluffs
Total storage	11.6			

¹Commissioned in summer 2006.

²Commissioned in fall 2006.

The City replaced the original Golf Course 1 and 2 tanks in 2005 because they were damaged by an earthquake decades ago, and experienced significant leakage.

It is also important to mention that the remaining Golf Course 3 and 4 tanks have an overflow elevation more than five feet lower than that of the new GC tanks 6,7,8,9. The City is considering using altitude valves at the old tanks so the new tanks will have full operational range without overflowing the old tanks.

Total storage volume represents about 80% of annual average day consumption. Under Black & Veatch design guidelines, storage should be sufficient to supply two times the difference between maximum day and peak hour water demands over four to six hours. Storage is replenished when demands drop below maximum day use rates.

Taking 2004 as a worst-case scenario, where the difference between maximum day and peak hour was about 10 mgd, and using six hours as the guideline, this would correspond to a requirement for about 5.0 mgd of system storage.



Storage Reservoirs – State Requirements

Utah DDW guidelines require equalization storage of 400 gallons per equivalent residential connection (ERC) in addition to any fire suppression and emergency uses.

Emergency needs are determined by the City, and fire suppression is determined by the fire marshal. Assuming one major four-hour fire of 3000 gpm would result in a fire suppression requirement of 0.7 MG.

Table 3.5 summarizes calculations for State-required equalization storage. Storage for indoor residential and ICI use is based on a rate of 400 gallons/ERC. Currently, ICI demand is about 90% of residential demand. City data suggests an average of 3.2 persons per ERC. For outdoor use, the State requires 2,848 gallons of storage per residential acre.

Table 3.5
Storage – Utah DDW Requirement for Equalization

Type of Use	gal/ERC	Population	ERC	Storage (MG)
Residential – indoor	400	48,194 ¹	15,060 ²	6.0
ICI	400		13,554 ³	8.6
Type of Use	gal/acre		acres	
Residential – outdoor	2,848		3,012 ⁴	5.4
Total Storage Requirement for Equalization				20.0

¹Year 2006 estimated population.

²Assumes 3.2 persons per ERC.

³Current ICI (including USU) demand = 90% of residential demand.

⁴Assumes average residential lot size of 0.2 acres.

Booster Pump Stations

Booster pump stations provide potable water to the USU, Castle Hills, Cliffside and Quail Bluffs zones. Table 3.6 summarizes information on these booster stations. From calibration modeling, it would appear that at some of these pump stations the pumps may be oversized, in which case operationally either valves are being pinched on the discharge side of the pump to reduce pressures, or they cycle more than necessary. Another possibility is that the pump impellers are worn, resulting in decreased output compared against the original pump curves.

Note that Cliffside I booster station is out of service and has been replaced by Cliffside II.



Table 3.6

Existing Booster Pump Stations

Well	No. Pumps	hp	Capacity (mgd)	Head (feet)
Golf Course 5	2	200	4.2	226
Castle Hills	2	50	1.0	200
Cliffside I	0	Pump station out of service		
Cliffside II	2	125	1.1	500
Quail Bluffs	2	100	0.9	206

PRV Stations

The network has a total of nine pressure reducing valve stations - three associated with the USU system, and the other six are not in operation due to public opposition some ten years ago when they were commissioned. These are described in Table 3.6. A tenth PRV station has been relocated between the GC 5 tank and the USU/Castle Hills area, and may be brought back on line in the near future.

Table 3.7

Existing PRV Stations

PRV Location	Size (in)	Setting (psi)	Operating?	From-To
Golf Course Tanks			Not yet	GC 5 tank-Castle Hills
14 th North 1350 East	8	54	Yes	Castle Hills-Upper USU
13 th North 12 th East	6	54	Yes	Upper USU-Lower USU
10 th North 8 th East	10	98	Yes	Upper USU-Lower USU
14 th North 330 East	10		No	Main zone
10 th North 270 East	12		No	Main zone
7 th North 270 East	24		No	Main zone
6 th North 270 East	12		No	Main zone
3 rd North 220 East	10		No	Main zone
Center Street 205 East	10		No	Main zone

Distribution Mains

Other than the DeWitt Springs pipeline, the most important mains are the 24" transmission mains from the Golf Course tanks. One of these mains stop at 8th East, while the other continues all the way to 1st West and ties into small-diameter pipes.



The Crockett, Center Street and 6th East wells are connected by 16" transmission mains; and the 6th East well ties into the second 24" transmission main. The Willow Park well only has 10" mains tying into the system, but there are some scattered segments of 18" transmission mains on the west side, attempts to implement the 1997 Water Master Plan.

Table 3.7 shows the distribution of network pipe by diameter as derived from the SCADA data. This does not include service connections and fire hydrant lines.

Table 3.8				
Existing Distribution Mains by Diameter				
Diameter (in)	Length (mi)	% of total	Length (mi)	% of total
36	0.01	0.0%	total 14-36"	total 14-36"
30	0.8	0.3%		
24	6.0	2.5%		
20	0.6	0.3%		
18	3.3	1.4%		
16	5.5	2.3%		
14	2.0	0.8%		
12	68.6	28.9%	total 8-12"	total 8-12"
10	28.6	12.0%		
8	42.5	17.9%		
6	52.9	22.3%	total 2-6"	total 2-6"
4	23.7	10.0%		
2	3.1	1.3%		
Total	237.7	100.0%		

While hydraulic modeling has shown very good delivery through most of the system, fire flow analysis has also highlighted the significant need to replace 2", 4" and even 6" pipes in order to meet fire flow needs.

Pipe replacement planning deals with issues associated with pipe age and materials. This is addressed in Section 6: Pipe Replacement Plan.



4.0 Hydraulic Model Development

4.1 Model Physical Facilities

The City of Logan has long recognized the importance of hydraulic modeling as a tool in planning improvements to water distribution systems.

Previously the City employed a model of the network in EPANET with over 1000 pipes and 800 junctions. This model showed only about 20% of the pipes, and was not based directly on actual maps.

Rather than merely update the existing model, the City agreed to convert to a GIS-based all-pipes network model. This required a significant effort and time for the City to update the network information in GIS, but provides a much more accurate hydraulic model.

Black & Veatch then brought these files into H2OMAP and tested the resulting network for connectivity in an iterative process to resolve discrepancies. The resulting all-pipes model for the existing system contained over 5,600 pipes and 5,200 nodes.

The hydraulic model employs the Hazen-Williams empirical equation to compute flows and head losses through the pipe network. The Hazen-Williams equation expresses pipe interior roughness in terms of a coefficient referred as the “C” value. The C value can be a function of pipe material, age, lining, cross-sectional area, amount of tuberculation, and other factors.

City of Logan operations staff report very little tuberculation or other signs of internal corrosion in the pipe network. A survey of water quality during the regulatory review confirmed the stability of the water. Therefore C values were related to pipe material.

Information was obtained on existing pump stations, tanks and PRV stations in order to set these up in the model. Information on some well pumps and booster stations was particularly sketchy because the pumps were installed more than 30 years ago.

Model physical facilities are assigned an installation date to keep modeling scenarios separate. This was particularly important since some tanks were taken out of service and others brought into service during the two years being considered for validation.



4.2 Population Apportionment and Demand Allocation

The City of Logan had ample evidence that population growth in the city regularly exceeds official Census projections. The City prepared their own geographically-related population projections, which were employed both for existing system and future system demand allocation.

These projections were then related to the official TAZ, triangulated to the individual demand nodes, and then consumption rates were applied to approximate the year 2005, 2010 and 2025 water demands.

Large users were individually allocated to the hydraulic model. Detailed billing records were available for USU, and these were assigned to the nodes nearest each metered location. Unaccounted-for-water was evenly distributed to model nodes.

All demands assigned to the hydraulic model represent average day demand conditions. Demand factors were assigned to the model for maximum day and peak hour, as well as the Extended Period Simulation (EPS) analyses used to calibrate the model. For the EPS validation analyses, the demand pattern employed for residential, ICI and large users was developed based on SCADA production and tank levels data.

EPS analyses are effective for calibrating hydraulic models because the cumulative effect of 24 steady state model runs helps to identify deviations that may require adjusting pump curves or pipe roughness coefficients so that the model will have predictive value. Taking into account the completeness and accuracy of SCADA data available, the Logan model can be considered adequately calibrated if pressures predicted by the 24-hour model runs are within 20 psi of the SCADA pressure records, and predicted levels for the tanks are within two feet of the SCADA tank level records for the larger tanks.

4.3 Model Validation (12th of July 2004)

Historical water use and available SCADA data were reviewed to select demand and operating conditions for EPS validation of the hydraulic model. Higher demands place greater stress on the distribution system, including operation of more pumps and wells, and generally represent optimal conditions for validating a hydraulic model.



Selection of Maximum Day for EPS Simulation

Two days were considered for model validation: July 12th, 2004 and July 29th, 2005. However, the second date was about the time the older leaking Golf Course tanks were being taken out of service and a new tank commissioned, so it was difficult to establish which tanks were actually in service on this day.

Therefore the 12th of July 2004 was selected, since it the data set was under defined operational conditions. This date also happened to include the peak hour demand for the year.

Quarter-hourly data on tank levels, and production and pressure data from DeWitt Springs and each of the four wells, was obtained from the SCADA system. The SCADA system had only daily production for the Willow Park well.

It was possible to see from quarter-hour pressure readings when the well was out of service. Based on the pump curve, a good approximation of production from Willow Park could be obtained.

Figure 2.6 (in section 2.3) shows the diurnal demands on the 12th of July 2004 based on the sum of production from DeWitt Springs and the four wells. However, this production data only provides a rough indication of actual demands.

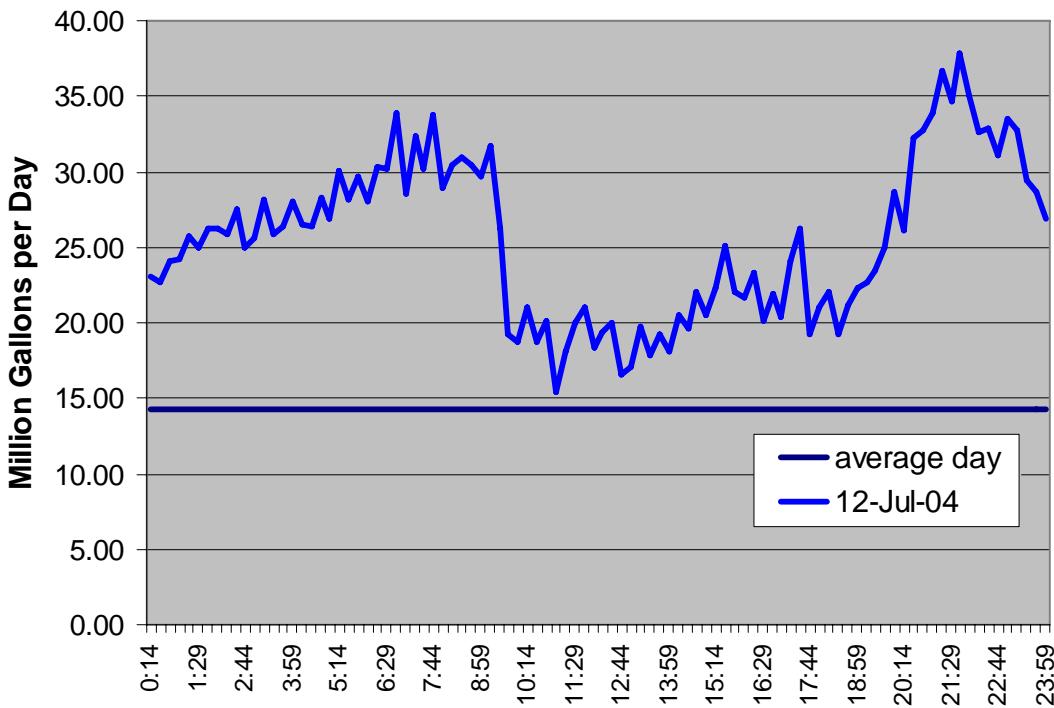
Matching production data with flows into/out of the reservoirs, and summing this provides a better picture of the actual demand curve for this maximum day. This is shown in Figure 4.1. This curve was used in the EPS model validation.

The high demands experienced during the early morning and late evening suggest considerable after-hours lawn irrigation. But even the lowest demands, which in this case were in the middle of the day, are much higher than the lowest demands on the maximum day in 2005, so again this day better represents a peak demand event than 2005 data.

Model validation simulated demand conditions, production data, and tank operation over a 24-hour period. Validation using EPS is more effective at producing a reliable model calibration, in part because errors will have a cumulative effect on model results. At the same time, calibration level of confidence depends on the accuracy of the operations data.



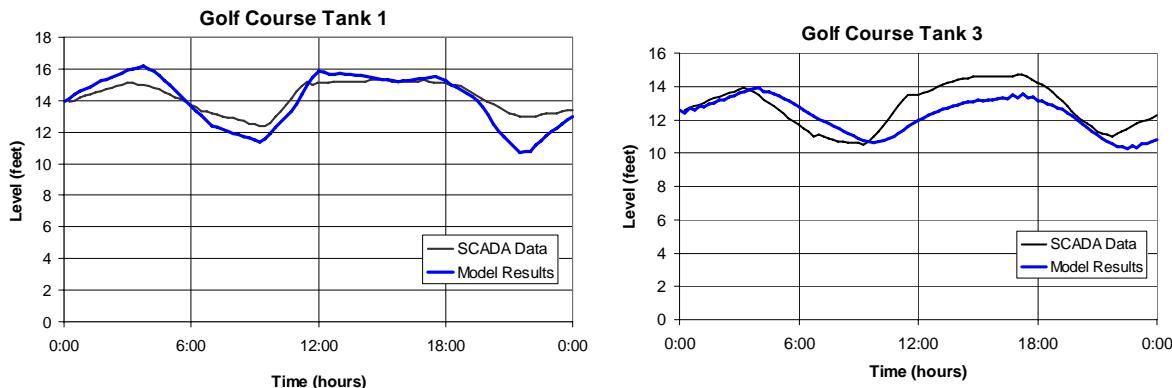
Figure 4.1
Logan Diurnal Demands on Maximum Day in 2004



Comparison of SCADA Data with Model Results – Main Zone

Model performance is assessed based on the ability of the model to accurately replicate changes in water levels in storage tanks, and pressures at the four wells. The following figures compare SCADA data on tank water levels with that predicted by the model.

Figure 4.2
Golf Course 1 and 3 Tanks Levels (July 12th, 2004)





As the graphs show, the model replicates closely the tank levels recorded in the SCADA system within two feet, and usually much less. Both graphs show deviations in tank levels toward the end of the 24-hour simulation within the range of acceptability. Golf Course 2, not shown, follows the same pattern as Golf Course 1. Golf Course 4, not shown, follows the same pattern as Golf Course 3.

The next set of figures compare production and pressures at the wells recorded by the SCADA system with those predicted by the model, and provide a further indication that the model calibrates well for the main pressure zone. The SCADA systems did not record quarter-hourly production data for the Willow Park well, but the pressure data predicted by the model closely matches that reported by SCADA, suggesting that the pump curve for this newer well is still fairly accurate.

Figure 4.3
Center Street Well Production and Pressure (July 12th, 2004)

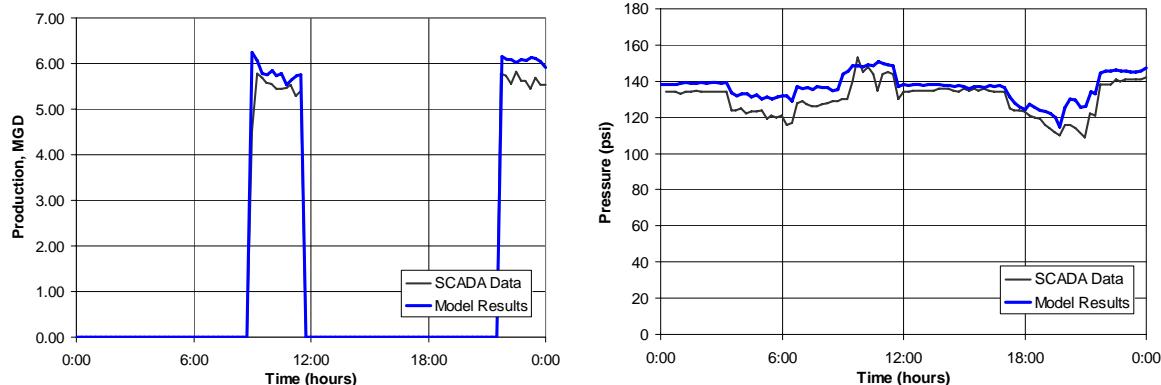


Figure 4.4
Crockett Well Production and Pressure (July 12th, 2004)

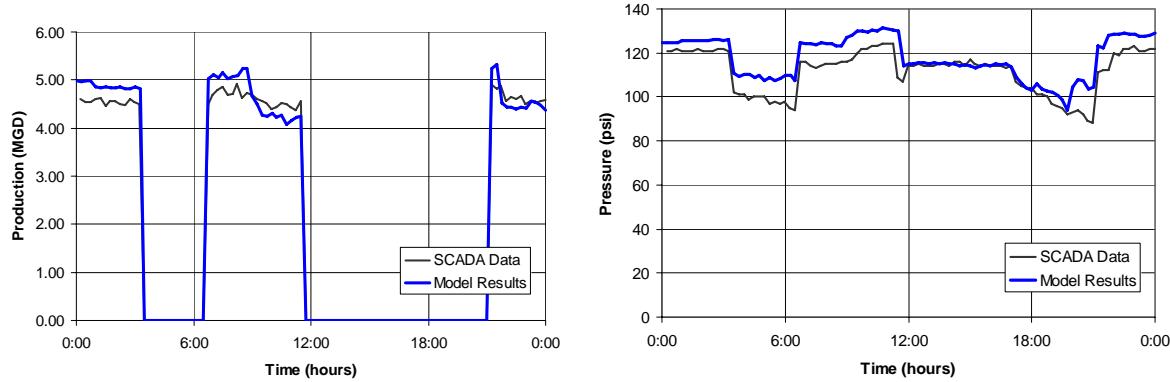




Figure 4.5
6th East Well Production and Pressure (July 12th, 2004)

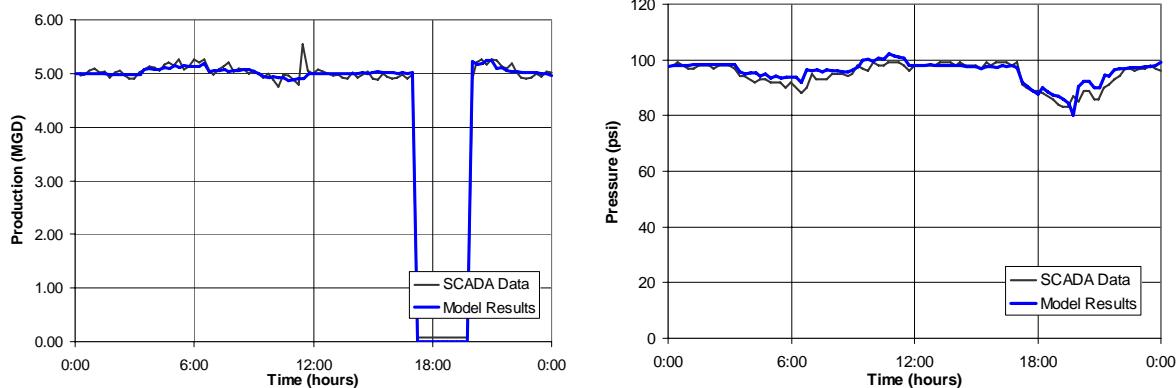
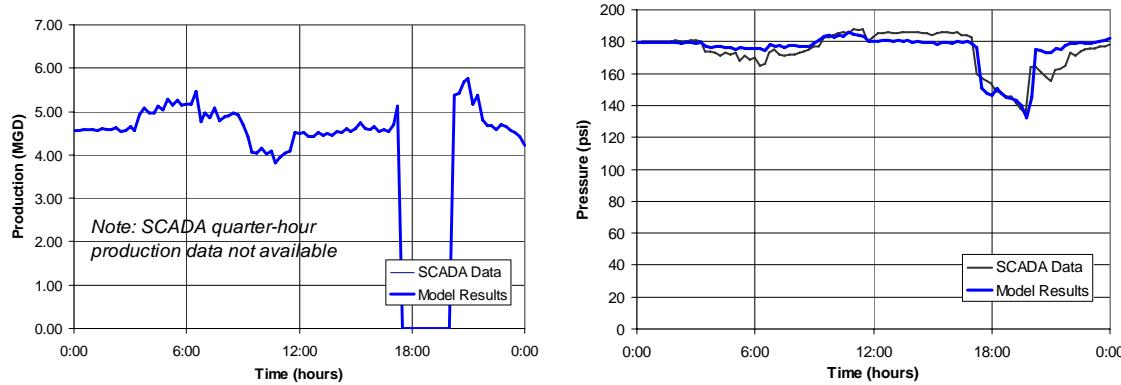


Figure 4.6
Willow Park Production and Pressure (July 12th, 2004)



The Center Street well was experiencing mechanical problems on the maximum day, and for this reason operated for only a few hours. SCADA data was not available for Willow Park production and had to be estimated using the pump curves. However, predicted well production for all wells was at all times within 10% of actual recorded production, and predicted pressures were at all times within 10-15 psi of recorded values.

Comparison of SCADA Data with Model Results – Smaller Zones

For the smaller zones, predicted tank levels followed closely SCADA operations data sufficient to indicate that these zones also calibrated well. The predicted curves would be smoother for the smaller zones if zone-specific demand data were available, but for the purposes of calibration the results are clearly predictive.



Golf Course 5 serves USU and Castle Hills. The predicted levels for Castle Hills deviate slightly over time. The pump curve had to be adjusted by a factor of 0.58 to calibrate this system. Using an even lower value would close the gap in deviation, but this is sufficient for calibration purposes. The discrepancy suggests that the pump curve is incorrect, the pump impeller is worn, a valve on the discharge side of the pump is used to throttle output, or a bypass valve is bleeding flow back to the lower pressure zone.

The model predicts levels for Cliffside and Quail Bluffs that agree well with SCADA data, even using unadjusted pump curves. Adjusting the curve for Quail Bluffs would further close the gap, but the results are sufficient for calibration purposes. The City may desire to verify pump operation and ensure valves are not leaking at this pump station.

Figure 4.7
Golf Course 5 and Castle Hills Tank Levels (July 12th, 2004)

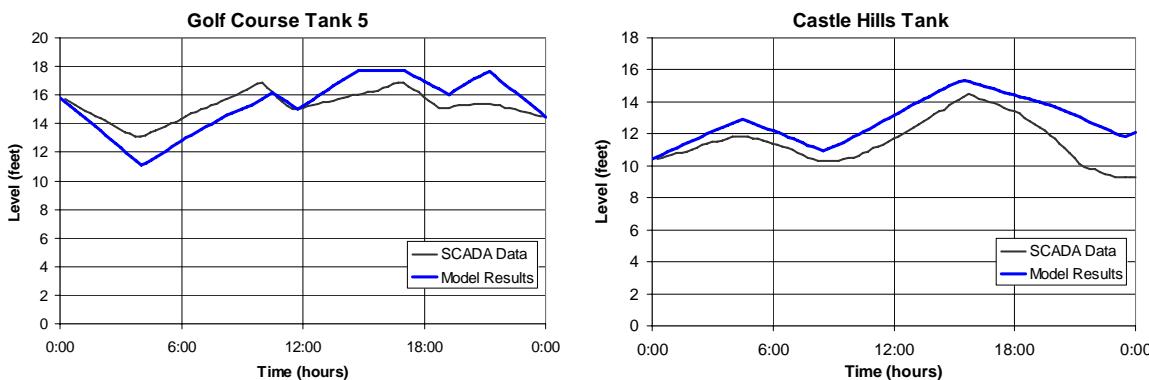
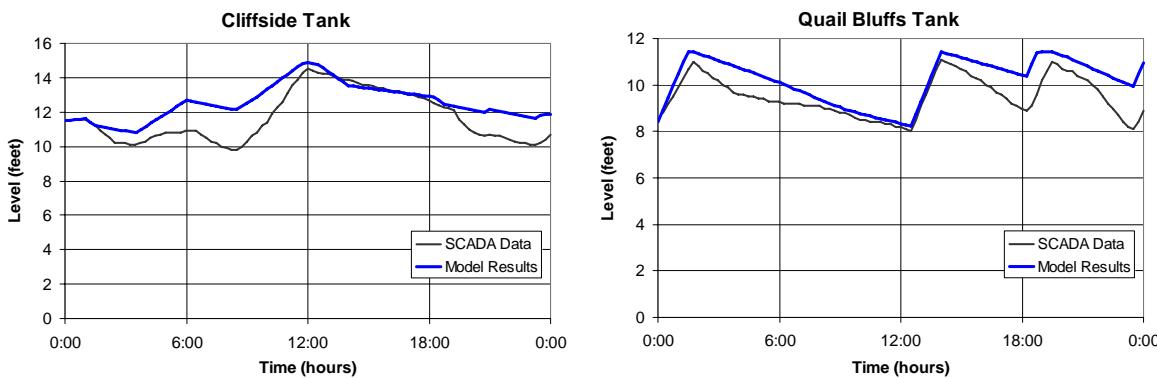


Figure 4.8
Cliffside and Quail Bluffs Tank Levels (July 12th, 2004)





4.4 Existing System Analyses

Existing System Analyses - Scenarios

The model was calibrated based on July 12th, 2004 because it was known exactly which Golf Course tanks were operable on this day, SCADA data was reliable, and this captured both the maximum day and peak hour for 2004, which were higher than 2005 values.

In recognition of the significant changes made to the Golf Course tanks area in 2005, where two tanks were demolished and four were constructed, it was decided to analyze the existing network based on 2006 conditions. At this point, the first two of four new Golf Course tanks were commissioned and the older tanks were decommissioned.

City staff identified the following scenarios for analysis, in order to better understand the existing system and define its vulnerability:

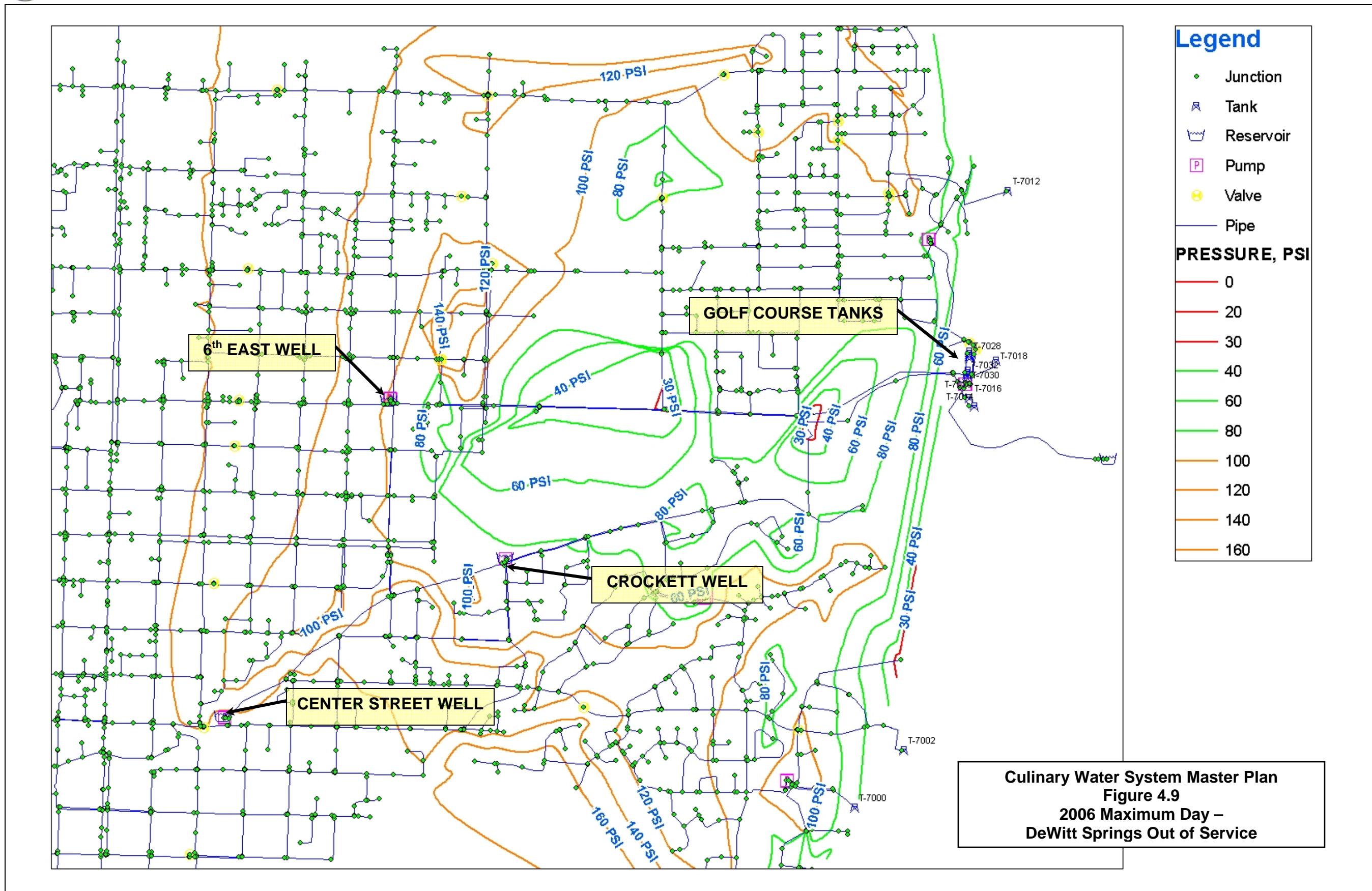
1. DeWitt Springs pipeline is out of service (major pipe failure or earthquake).
2. Golf Course tanks are out of service (earthquake).
3. Willow Park well is out of service (operational, power or pipe failure).
4. Multiple fire flows in the Central Business District (earthquake, plane crash) with fire flows of 1500 gpm each at three nodes.

Initially analyses were to include both maximum and average day conditions, but the results did not differ significantly, and the maximum day analyses provided an adequate basis for evaluating the strengths and weaknesses of the distribution network. Results from each of these scenarios on a maximum day are discussed below.

DeWitt Springs Pipeline Out of Service

In this scenario, the total demand is 17.9 mgd – losses on the DeWitt pipeline are not included in this run. Assuming that all four wells are in service, they would be able to produce 19.7 mgd and thus keep the Golf Course tanks from draining. Modeling predicts minimum pressures just above 20 psi, while maximum pressures are just above 190 psi.

Figure 4.9 shows pressure contours for the east side of the service area, where most low pressures will occur. This shows only small areas with low pressure in the USU area; and it shows that below the bench pressures quickly exceed 100 psi. Pressures gradually rise westward across the city, reaching 190 psi.





The network is able to sustain operation with the largest single supply source out of service, and with low pressures only at a few locations nearest tanks. This owes largely to the fact that replacement of the leaking reservoirs has provided a margin of safety in the current supply. The results from this run and the next runs do not differ substantially.

Golf Course Tanks Out of Service

This scenario assumes the Golf Course tanks are out of service due to an earthquake. Since they are only 400 feet east of the fault line, this is not an unreasonable assumption.

Demands are 20.6 mgd, but this value includes the USU and Castle Hills zones and leakage in the DeWitt Springs pipeline.

This scenario is more critical, because Golf Course tank 5 serves the USU and Castle Hills zones. With the tanks out of service, these zones will have no water supply.

While USU may have an emergency connection to the network, and thereby can maintain at least some pressure, the Castle Hills zone does not have that option.

Excepting the USU and Castle Hills zones, where the model cannot be used when there is no supply, minimum pressures in the network are predicted at just above 20 psi, while maximum pressures are at 215 psi. The four wells are all operating, but do not have to satisfy demand of the USU and Castle Hills zones, so pressures are higher in this run.

Willow Park Well Out of Service

This scenario assumes that all other supply sources are operating, but that the Willow Park well is out of service. The intent was to determine if pressure could be maintained in the southern reaches of the main pressure zone.

Demands are 20.6 mgd and the tanks are still filling. The model predicts minimum pressures of 20 psi and maximum pressures of 184 psi, very similar to the first scenario. Since supply capacity exceeds maximum day demands, the main problem with the network is the high pressures in the lower areas.

It appears that the other supply sources, when all operating and without the former leakage at the reservoirs, can maintain pressures in the network on a maximum day when Willow Park is out of service.



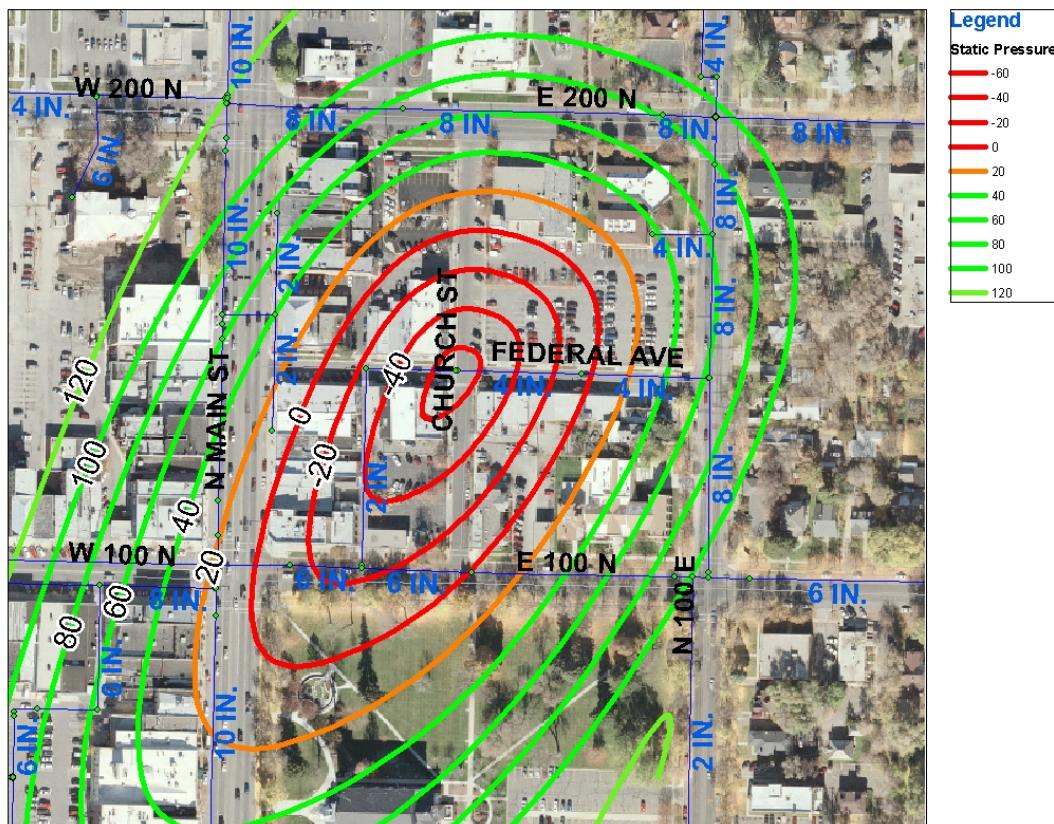
Fire Flow in Central Business District

This scenario considers a major fire in the center of the city, requiring three simultaneous fire flows of 1500 gpm each. The intent was to see how the network would respond. All four wells and the DeWitt Springs pipeline are in service in this run.

One fire flow was on a 4" line at Church Street and Federal Avenue. This resulted in negative pressures due to the small diameter – only one block away pressures were about 110 psi. The resulting pressure contours are shown in Figure 4.10.

The other two fire flow locations were located on larger lines and did not cause significant pressure drops. The network maintained maximum pressures above 180 psi, and minimum pressures at high points in the network remained almost unchanged.

Figure 4.10
2006 Maximum Day – Fire Flow Pressures at Church Street/Federal Avenue





Existing System Analyses – Conclusions

All of these runs underline the benefit that replacing the leaking older Golf Course tanks has had on the overall supply. The DeWitt Springs Pipeline and the four wells currently have a combined capacity of about 30 mgd, and have historically provided up to 35 mgd.

But implicit in this is the importance of a stable supply. If both the DeWitt Springs pipeline and just one of the four wells were taken out of service, it could be very difficult to supply maximum day demands and maintain system pressures.

It is noteworthy that several pressure zones have only one supply line. These include Castle Hills, and the Cliffside-Quail Bluffs zones. It is worth considering if emergency connections should be provided to these zones, as USU reportedly has to the main zone.

The fire flow run highlights the fact that the many smaller diameter pipes in the network cannot provide reliable fire flow. One-third of the network pipes are 6" diameter or smaller; one-tenth of the network pipes are 4" or smaller.

The modeling scenarios also highlight the consistently, extremely *high* pressures experienced in lower parts of the network. With most of the city operating as a single zone, and range in elevation of over 400 ft between the highest and lowest areas in the main pressure zone, high pressures in the lower areas are necessary to keep sufficient pressure in the upper areas of the zone.

These high pressures represent:

- A risk to City staff during maintenance and repair of the network.
- A risk to public safety and property when pipe failures occur.
- A stress on the pipe network that reduces pipe lifetime, increases the risk of failure, and increases the overall maintenance and renewal cost to the City.
- An energy requirement for pumping from the wells into the network that is more than twice what would be necessary in a normal network.
- Constant complaints from residents regarding damage to sprinkler systems, house PRVs, and service and internal pipes due to high pressures and water hammer.

For this reason one of the most important steps the City can take is to divide the main zone into two or more pressure zones each with lower service pressures. And another needed step would be to replace smaller diameter mains to improve fire flow capacity.



The City attempted to divide the network into pressure zones a decade previously. However, there are advantages now that were not present at that time:

- Pressure can be maintained in the southwestern part of the network more easily now that the Willow Park well is connected to the network.
- There are more large mains conveying water to the west and south sides of the network so that pressure distribution is more even.
- The proposed arrangement for pressure zoning (described in detail in section 4.6) will, once completely phased in, reduce energy consumption by more than 50% and still maintain stable pressure in the lower zone(s).

4.5 Level of Service Goals

With the findings of the existing system analyses, and a review of State requirements for water systems, Level of Service Goals were proposed for the future system. The following table summarizes some of the State DDW requirements drawn from R309-510 and R309-550.

Table 4.1
State of Utah DDW Requirements

Category	Requirement
Service Pressures	<ul style="list-style-type: none">• Minimum 20 psi, including peak day with fire flow• Normal working pressure 40-60 psi• Use pressure devices when exceeds 80 psi
Minimum Main Size	<ul style="list-style-type: none">• Minimum size 4-inch diameter• Minimum size 8-inch diameter when serving hydrant• Velocity should not exceed 5 fps
Fire Hydrants / Fire Flows	<ul style="list-style-type: none">• Average spacing of hydrants 500 ft• 1000 gpm for single/duplex homes up to 3600 ft²• 1500 gpm or greater for buildings• Length of fire 2-4 hours for buildings
Storage	<ul style="list-style-type: none">• Equalization storage 400 gallons per ERC for indoor use• Equalization storage 2,848 gallons / irrigated acre
Dead End Mains	<ul style="list-style-type: none">• Avoid where possible• Provide fire hydrants or other flushing at dead ends
Severely Corrosive Soils	<ul style="list-style-type: none">• Plastic pipe is recommended



Currently, the City does not comply with several aspects of the State DDW rules:

- Service pressures in the City network are much higher than advised by DDW.
- Parts of the older network have fire hydrants on mains smaller than 8" diameter.
- Storage requirements may be considerably higher than currently provided.

It should be noted that DDW requirements for storage, as currently interpreted, are considerably higher than would be required in other states. This owes primarily to the high outdoor water use in Utah, and the fact that the DeWitt Springs and wells, which would be given storage credit in other states, are not credited towards storage by DDW.

A calculation of storage requirements is shown for 2006 and 2025 in Table 4.2.

Table 4.2			
Equalization Storage Requirements, 2006 and 2025			
Category	Population	ERC	Storage Required
Year 2006			
Residential – indoor	48,194	15,060 ¹	6.0 MG
Residential - outdoor		0.2 acres / ERC	8.6 MG ²
Inst.-ind.-commercial		13,554 ³	5.4 MG
Total Equalization Storage Required in Year 2006			20.0 MG
Year 2025			
Residential – indoor	74,441	23,263 ¹	9.3 MG
Residential - outdoor		0.2 acres / ERC	13.3 MG ²
Inst.-ind.-commercial		20,937 ³	8.4 MG
Total Equalization Storage Required in Year 2025			30.9 MG

¹3.2 persons per ERC.

²Zone 4 in DDW table on irrigated crop consumptive use: 2,848 gallons storage per acre.

³ICI consumption = 90% of residential consumption, therefore ICI ERCs = 0.9 residential ERCs.

The year 2006 requirement is almost twice the actual year 2006 storage of 11.6 MG, and does not include fire suppression and emergency volumes. The increase from 2006 to 2025 in population and industrial/commercial/institutional growth results in a 50% increase in storage requirements, or almost triple year 2006 storage.

Minimum level of service goals are proposed in Table 4.3 below.



Table 4.3

Minimum Level of Service Goals

Category	Goal
Service Pressures	<ul style="list-style-type: none">• Minimum 20 psi for peak hour• Minimum 30 psi for maximum day with fire flow• Minimum 40 psi for maximum day• Maximum 100 psi where possible
Minimum Main Size	<ul style="list-style-type: none">• Minimum size 8-inch diameter to serve hydrants
Fire Flows	<ul style="list-style-type: none">• 1000 gpm for single/duplex homes up to 3600 ft²• 1500 gpm for buildings• Length of fire 2-4 hours for buildings
Storage	<ul style="list-style-type: none">• Equalization storage 400 gallons per ERC for indoor use• Equalization storage 2,848 gallons / irrigated acre
Dead End Mains	<ul style="list-style-type: none">• Avoid dead end mains• Provide fire hydrant only for flushing at dead ends
Corrosive Soils	<ul style="list-style-type: none">• Use plastic pipe

The City would like to keep minimum pressures above 40 psi. Lower pressures are acceptable in mains near the tanks or high points on transmission mains. However, there are several locations, including an athletic field on the north end of the Quail Bluffs zone, where the tank elevation would have to be adjusted or booster pumps provided if higher service pressures were desired.

4.6 Year 2025 Hydraulic Analyses and Recommended Improvements

Development of Year 2025 System – Expansion of Service Area

The City identified future development areas which in effect double the year 2025 service area. Some areas are scheduled for industrial and commercial development, others for residential. Much of the future far west side is planned as low-density residential/environmental areas. Buildout for most of these areas will occur after 2025.

The City-prepared population and employment projections for 2030 were based on land use zones and phased development of the future service areas established by the Community Development Department. Based on this information, Black & Veatch calculated population and employment for the year 2025, incorporated this into the TAZ, applied per capita water demands, and allocated these demands to the nodes.



The year 2025 network in the western annexation areas was laid out based on a review of when and where the projected population growth is predicted to occur, and represents an attempt to maintain a looped system. A minimum pipe diameter of 12" was employed for all new pipes in these areas instead of the traditional 8" diameter.

The actual development of the network over the next 20 years, and selection of diameters, will depend in large part on the timing and magnitude of individual developments approved by the City. If large developments are permitted on branch lines, the City will need to determine by site-specific modeling if these branch lines should be oversized or if it will be more cost-effective to loop the system in this area.

Figure 4.11 shows the existing network and in red the pipes, tanks and well to be included in the network by 2025. The new pipes amount to 69 miles of mostly 12-inch pipe – a 40% increase in overall network length to 238 miles.

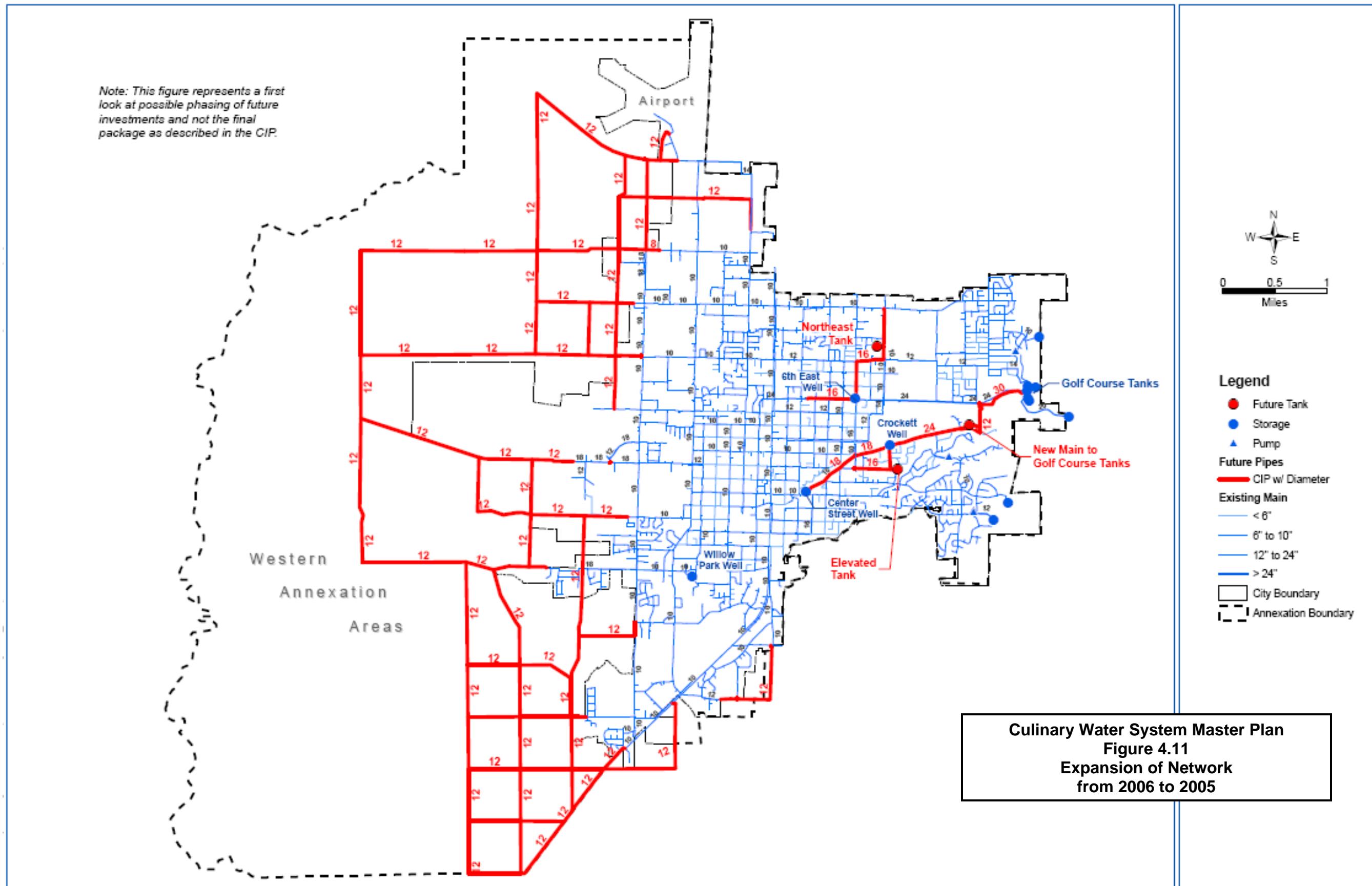
This figure illustrates that the service area will double during this period, with the need for new network piping occurring primarily west of 10th West.

Development of Year 2025 System – Pressure Zoning

Based on State DDW requirements for lower system operating pressures, and risks to City staff and public safety, the most important improvement to the network during the overall master planning period would be to restructure the main pressure zone to create two or more pressure zones with lower service pressures.

The existing main pressure zone could be divided into three pressure zones:

- The Upper Zone, primarily served by DeWitt Springs, to include a future well planned on the east side. Where DeWitt Springs supply exceeds water demands in this zone, the additional water would be conveyed by PRV stations to lower zones.
- The Central Zone, served by DeWitt Springs via PRV stations, but also served by the Center Street, Crockett, 6th East and Willow Park wells, which would pump directly to this zone. Ground or elevated storage would also be needed to reduce pump cycling and provide stable pressure in this zone.
- The Lower Zone, including the western annexation areas, to be served from the Central Zone by a series of PRV stations.





Ideally, the boundary between the Upper and Central zones would be in a north-south line at about 6th East, and the boundary between the Central and Lower zones would be in a north-south line just past 6th West. This would give maximum static pressures in the Upper Zone of about 105 psi, and allow reducing pressures in the Central and Lower Zones to less than half the current supply pressure.

However, the City already has six PRV stations in a north-south line between 2nd and 4th East, just three to four blocks west of the ideal boundary line between the upper and lower zones. With little effort the settings could be changed at these PRV stations and they could be made operational to establish the pressure zone boundary between the Upper Zone and Central Zone.

Resulting maximum static pressures in the Upper Zone would be almost 140 psi the first one or two blocks east of the PRV stations; but pressures in the lower zones could be adjusted to more closely match DDW requirements.

There is also a large neighborhood east of the Center Street well which due to its low elevation should belong to the Central Zone to reduce the current very high service pressures. The equipment at the Center Street PRV station could be moved to a new location farther east to further define the boundary in this area. Phasing of these changes is discussed in Section 8 Capital Improvements Plan.

If the existing PRV stations are used to define the boundary between the Upper and Central Zones, then the boundary between the Central and Lower Zones can be moved to 10th West. This area is less developed, so fewer PRV stations would be necessary to establish the Lower Zone, and PRVs could be installed at the same time as new mains are installed in conjunction with developments further west.

In addition to dividing the main zone into three pressure zones, there is a need to provide small PRV stations at two locations in the Cliffside zone. There are neighborhoods built into a wash where lower elevations result in pressures above 150 psi. PRV stations were therefore incorporated into the model on Mountain Road at the intersection of Eastridge Drive; and on Red Fox Trace at the intersection of 14th East.

The resulting proposed static pressures for each pressure zone are shown in Table 4.4.



Table 4.4

Year 2025 Pressure Zones

Pressure Zone	Ground Elevation	Static Level (MSL)	Static Pressures (psi)	Controlling Reservoir
Upper Zone	4550-4840	4875	15-140	GC 3,4,6,7,8,9
Central Zone	4450-4550	4680	56-100	new tank(s)
Lower Zone	4430-4460	4570	48-60	new PRVs
Upper USU	4750-4846	5057	45-132	GC 5
Lower USU ¹	4650-4846	4930	36-122	PRVs
Castle Hills ²	4770-4940	5055	50-123	Castle Hills
Cliffside ³	4790-5016	5065	20-120	Cliffside
Quail Bluffs	4890-5102	5149	20-112	Quail Bluff

¹Upper USU and Lower USU may be operated together, but pressures would be extremely high.

²Castle Hills may be operated on the GC 5 level together with USU.

³PRV stations added to lower pressures in two small neighborhoods.

Hydraulic modeling of the year 2025 network indicated overall good water distribution. Other than the small-diameter pipes, which are a fire flow issue, there was only a small area in the “Island” with systematically low pressures. This was resolved in the model by replacing the 4-inch line from 7th North down to Canyon Road with a 12-inch line all the way to 1420 East. City staff confirmed the observation from the model, and staff has been considering the same improvements to supply in this area.

Hydraulic modeling shows that the wells can provide the necessary pressure for the Central Zone. However, experience also shows system storage is useful to provide stable pump operations and prevent pump cycling with accompanying reduced pump life. It is recommended to add storage into the Central Zone for this purpose.

The system was modeled looking at two possibilities:

- A 2 MG ground storage tank located in the area of 8th East north of 10th North.
- A 0.5 MG elevated storage tank located on the east side near the Logan River.

The model did not indicate a particular preference – both locations would be able to provide pressure stability. Therefore the decision of where to locate storage, as well as the amount of storage required, is primarily an operational consideration.



Development of Year 2025 System – Reducing Vulnerability

Restructuring the main network into zones provides several opportunities for reducing system vulnerability and increasing operational flexibility.

On the supply side, City staff has been planning to install an additional well to reduce vulnerability in the event of failure of the DeWitt Springs source. This well would be located on the east side of town, though the specific location has not yet been decided.

On the delivery side, should the DeWitt Springs source be out of service, there would be a need to deliver water to the Upper Zone, including USU and Castle Hills zones. For this reason City staff has considered connecting the future well via a dedicated transmission main directly to the Golf Course tanks, where chlorination can also be provided.

Another possibility for operational flexibility and reduced vulnerability presents itself. The Center Street, Crockett and 6th East wells are situated within blocks of the proposed boundary between the Upper and Central zones. Each of these wells is intended to supply water to the Central and Lower zones. However, they could be modified to also be able to provide water to the Upper Zone should the DeWitt Springs supply be compromised.

In this option, each new well pump would be designed to provide pressure to the Central Zone. Then a second pump near each well could be operated to boost flows into the Upper Zone should emergency conditions warrant.

The Center Street and Crockett booster pumps could use the proposed transmission line to supply the Golf Course tanks. The 6th East booster pump would convey flow directly into the Upper Zone via the existing 24-inch transmission main. Chlorine dosing facilities should be considered at the wells to maintain disinfectant residual when the Golf Course tanks of DeWitt pipeline is out of service.

Also, the future tank to which 6th East pumps could be connected directly or indirectly to the hospital to ensure a secure supply for emergency conditions.

Year 2025 System Analyses – Scenarios

The City of Logan considered that in the case of earthquake, worst-case scenarios may involve failure of *two* supply sources simultaneously on a maximum day. This is not unrealistic. The scenarios identified for maximum day vulnerability analyses included:



1. DeWitt Springs pipeline **and** Golf Course tanks out of service.
2. Center Street well **and** 24" transmission main (above 6th East well) out of service.
3. 6th East **and** Crockett wells out of service.

The year 2025 network was also analyzed for peak hour demands, and fire flow analyses were carried out in order to identify any system deficiencies that may not have become apparent with the other analyses.

DeWitt Springs Pipeline and Golf Course Tanks Out of Service

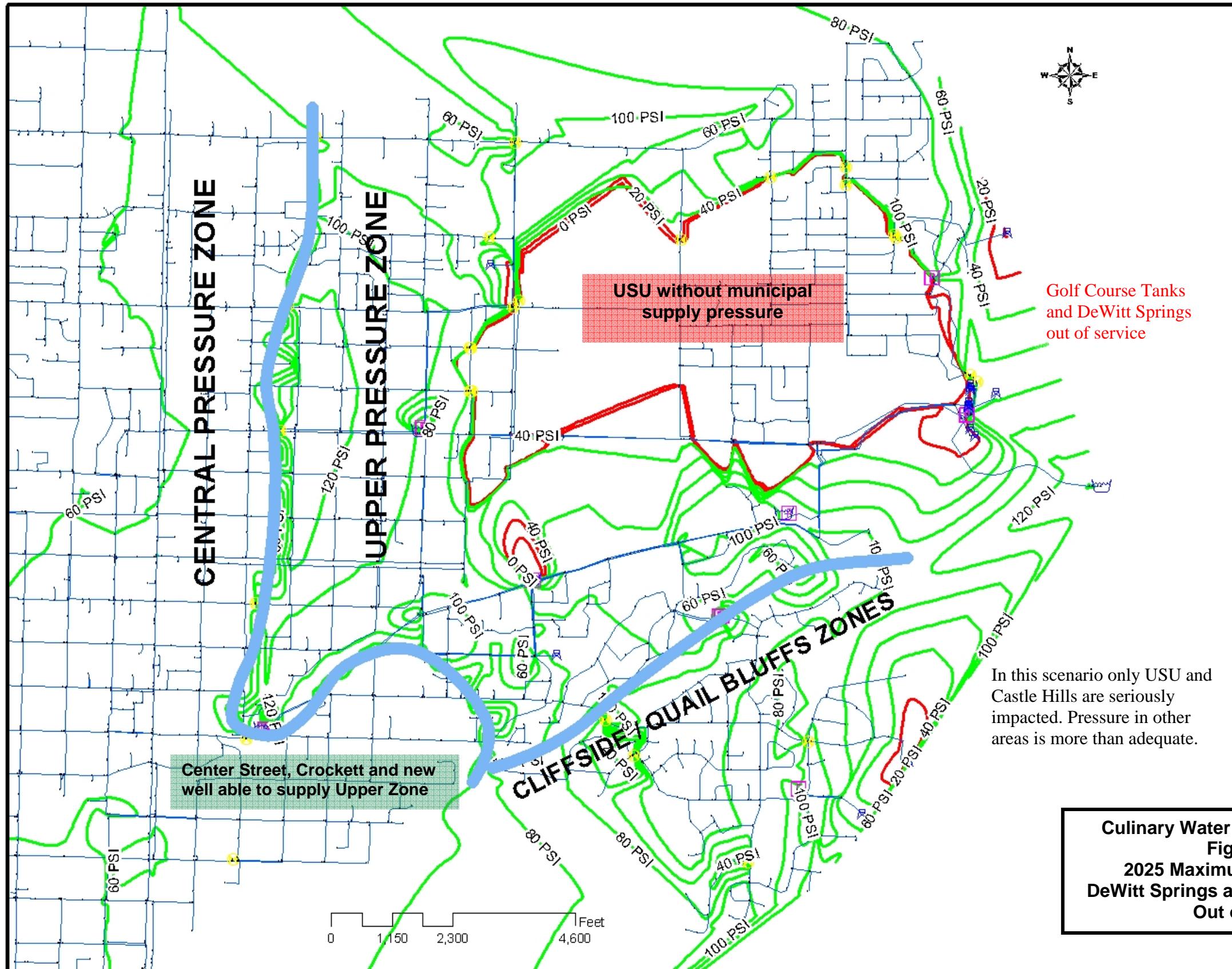
In this emergency scenario, it is assumed that the City has reverted to turning on the booster pumps at Center Street, Crockett and 6th East wells, and the new east well is operating, in order to maintain pressure in the Upper Zone. Thus this scenario primarily looks at impacts on the Upper Zone.

Together with the planned future well on the east side, the Center Street and Crockett wells convey water to a junction on the 24" transmission main downstream of the Golf Course tanks via the proposed dedicated transmission line.

Since the Golf Course 6-9 tanks supply the Golf Course 5 tank, which in turns supplies the USU and Castle Hills zones, under this emergency scenario there is no municipal supply to any of these areas. Unless USU can activate an emergency connection to the Upper Zone, or has sufficient capacity from its own well, it will not be possible to maintain pressure in the USU zone. Castle Hills has no other connections to the network but GC5, and will therefore be completely without water once the 0.5 MG Castle Hills tank is drained down.

Figure 4.12 shows the resulting pressures in the east/northeast part of the city. The model cannot calculate pressures without flow into an area. To be able to run the model, all pipes into the USU zone were closed but one 4" line. This allowed the model to run, but resulted in the negative pressures throughout the USU service area.

Castle Hills still shows positive pressures in the model, but this is because at the moment of failure the 0.5 MG Castle Hills tank is still able to provide water to this area. However, the tank is draining and there is no flow coming in to replenish the tank.



Legend

- Water Tank
- Pump Station
- Valve Station

Pressure

0
20
40
60
80
100
120
140

Golf Course Tanks
and DeWitt Springs
out of service

In this scenario only USU and Castle Hills are seriously impacted. Pressure in other areas is more than adequate.

Culinary Water System Master Plan
Figure 4.12
2025 Maximum Day Pressures
DeWitt Springs and Golf Course Tanks
Out of Service



With a local maximum day demand of almost 0.4 mgd, Castle Hills could maintain pressure for just over one day. However, if supply has been interrupted due to an earthquake, there may be significant leaks in this zone that straddles the fault line, and pressure cannot be maintained for long without major leak isolation.

Table 4.5 shows the results from this scenario run. With the exception of the USU zones, the rest of the network can function fairly well, primarily by providing booster pumping from the wells to the Upper Zone.

Table 4.5 Year 2025 Maximum Day Model Results: DeWitt Springs Pipeline and Golf Course Tanks Out of Service			
Pressure Zone	High Press. (psi)	Low Press. (psi)	Comment
Upper Zone	131	28	low pressure adjacent to 24" main at high point near tanks
Central Zone	83	41	
Lower Zone	83	47	
USU Zones	-	0	own well may provide pressure
Castle Hills	107	48	0.5 MG tank only supply
Cliffside	120	30	low pressure near tank
Quail Bluffs	120	24	low pressure near tank and at one high point

Center Street Well and 24" Transmission Main Out of Service

This scenario primarily looks at impacts on the Central Zone, where the Center Street well is out of service, and supply from the Upper Zone via the 24" main is compromised immediately upstream of the 6th East well. All the other wells and tanks are functioning.

Table 4.6 shows the results from this scenario run. Essentially this run shows that the network is able to handle the supply restrictions quite well, in part because laterals distribute water from the Upper Zone upstream of the PRV stations to the Central Zone. Pressures are good in most of the zones even though this occurs under maximum day conditions.

Residual pressures in the Upper Zone are already above 120 psi in the lower reaches, so low pressure problems would not be expected here.



However, should the failure in the 24" main occur at a location farther east (where there are fewer laterals to spread the flow), this could conceivably result in some localized pressure problems in the Central Zone.

Table 4.6
Year 2025 Maximum Day Model Results:
Center Street Well and 24" Transmission Main Out of Service

Pressure Zone	High Press. (psi)	Low Press. (psi)	Comment
Upper Zone	118	23	low pressure adjacent to 24" main at high point near tanks
Central Zone	84	43	
Lower Zone	84	48	
USU Zones	134	63	
Castle Hills	122	51	
Cliffside	120	28	low pressure near tank
Quail Bluffs	120	24	low pressure near tank and at one high point

6th East Well and Crockett Well Out of Service

This scenario looks at the consequences of having two supply sources out of service in the Central Zone. Table 4.7 shows the modeling results for this maximum day event. As with the other scenarios, there are high pressures in the Upper Zone, but no significant low pressures in any of the zones, and very little differences in modeled pressures.

Table 4.7
Year 2025 Maximum Day Model Results:
6th Street and Crockett Wells Out of Service

Pressure Zone	High Press. (psi)	Low Press. (psi)	Comment
Upper Zone	126	23	low pressure adjacent to 24" main at high point near tanks
Central Zone	85	44	
Lower Zone	86	50	
USU Zones	134	63	
Castle Hills	122	51	
Cliffside	120	28	low pressure near tank
Quail Bluffs	120	24	low pressure near tank and at one high point



Peak Hour Analysis

The year 2025 peak hour analysis assumes a peak hour to average day factor of 2.4; and all wells, tanks and springs are available to supply peak hour demands of 39 mgd.

The peak hour demands result across the board in lower peak pressures in the pressure zones, but there are very few pressures actually below those desired in the service level goals. Figure 4.13 shows the east side of the network, while Table 4.8 summarizes and comments on the results from this modeling run.

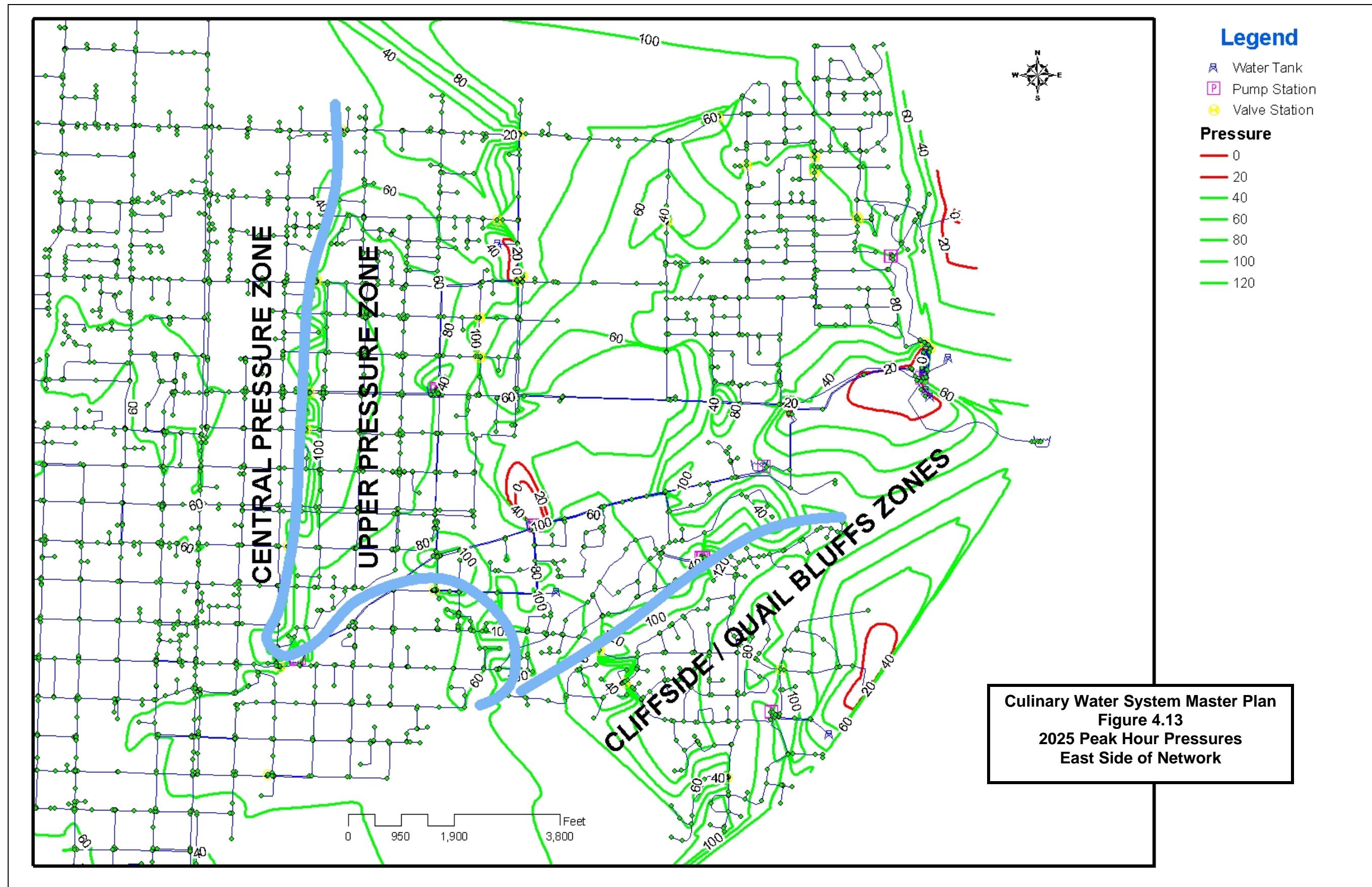
It should be pointed out that earlier modeling indicated problems with high pressure in Cliffside, low pressure in the Island, and other anomalies which have been corrected when setting up the year 2025 model. Therefore many of these pressure problems do not appear in the tables and figures presented in this report.

Table 4.8
Year 2025 Peak Hour Model Results

Pressure Zone	High Press. (psi)	Low Press. (psi)	Comment
Upper Zone	113	17	low pressure at high points near tanks, 24" main, and high point at closed USU valve
Central Zone	76	30	
Lower Zone	70	18	low pressure on 2" private line at composting facility; otherwise lowest pressure is 50 psi
USU Zones	116	30	
Castle Hills	119	49	
Cliffside	118	28	low pressure near tank
Quail Bluffs	118	22	low pressure near tank and at one high point

System-Wide Fire Flow Analysis

In this scenario, H2OMAP takes all the service nodes and runs two types of fire flows on each node individually – essentially several thousand runs in the present case, where fire flows were run for present and future system without improvements except for system expansion. This is one of the tools used to find water delivery problems that may not have appeared in the maximum day and peak hour scenario runs. Fire flow is not applied to large transmission mains, since these already have capacity well beyond the 1000-1500 gpm that might be desired.





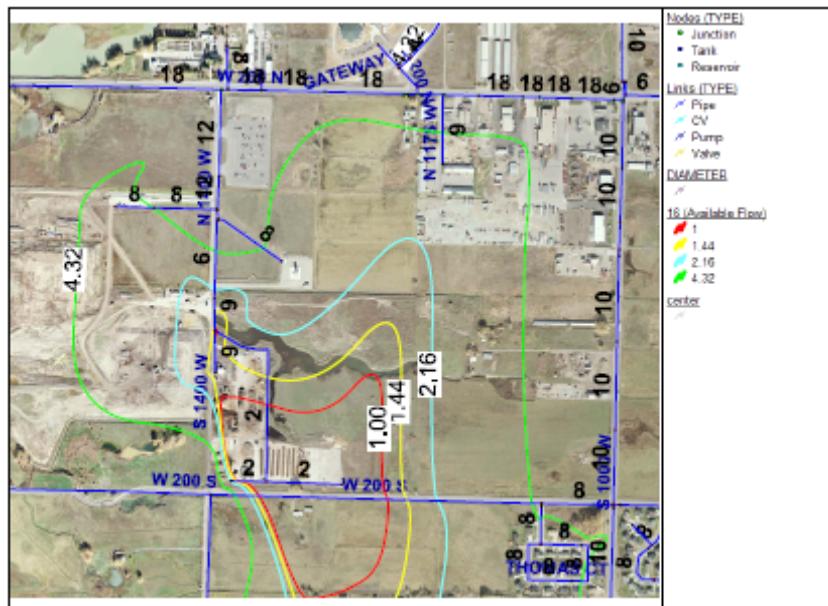
In the first type of run (Fireflow) a fire flow of 1500 gpm was assigned to every node, then assuming a minimum pressure of 20 psi the actual flow obtainable at that location was calculated. This can then be used to develop contours so that deficiencies are visible.

The second type of run (Fireflow Design) takes analysis one step further and calculates fire flow available at each node while maintaining at least 20 psi at critical nodes *elsewhere* in the network. Contours can then be developed to show deficiencies.

The model outputs for the first case, expressed in contours, showed the expected result that the smaller diameter pipes are the primary restriction. This included for example a 2" private line to the composting facility on the west side. While this may be a private line, it is still worth considering that a composting facility represents a fire hazard, and that it would be best to have sufficient fire flow capability nearby.

Fire flow results at this location are shown in Figure 4.14, where 1.44 mgd = 1000 gpm, 2.16 mgd = 1500 gpm, and 4.32 mgd = 3000 gpm. This is shown with the aerial background to help see the composting windrows.

Figure 4.14
2025 Fire Flow Analysis: the Composting Facility on West Side

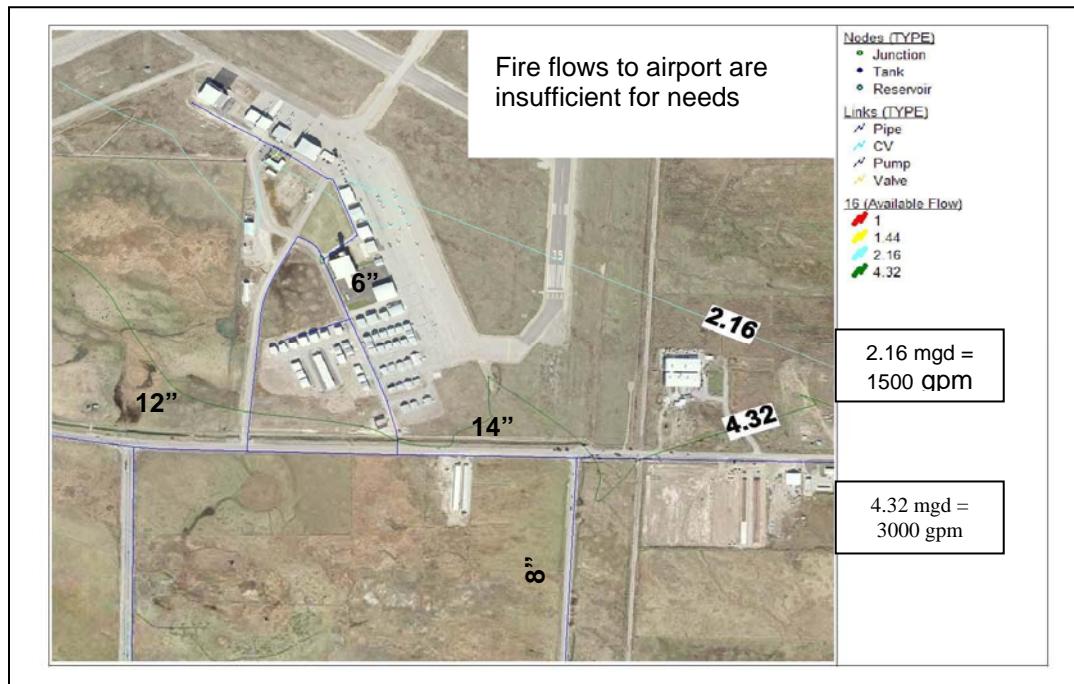




The model outputs for the second case covered much larger areas, but closer inspection of the critical nodes impacting the analysis shows that these were usually at high points in the network, such as immediately downstream of a closed valve (14th North and 8th East), that did not represent problems that needed correcting, and therefore the second run gave little additional insight into network limitations.

Fire flow analysis also showed the need for a second line to the airport, as contemplated by City staff. Figure 4.15 shows results for this area, with future pipelines in red.

Figure 4.15
2025 Fire Flow Analysis: the Airport

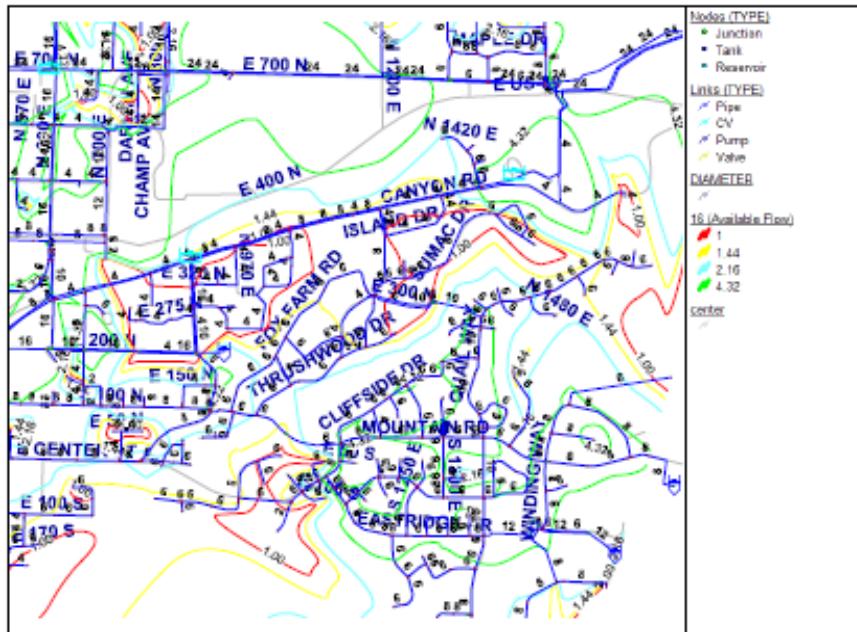


Ideally, the airport would maintain local pressurized storage for emergencies. Even with a 12-inch line added in the model, fire flow for areas closest to the airport is only 1500 gpm, but already 3000 gpm at the intersection with 25th North. If additional storage is not provided, the City will need to provide larger supply capacity to this area.

Fire flow analysis helped to highlight that it would be difficult to supply the “the Island” without a 12-inch line. But even with this line, the small-diameter pipes represent a risk that the City will not be able to provide sufficient fire flow in an emergency. Figure 4.16 shows results for this area.



Figure 4.16
2025 Fire Flow Analysis: the “Island” Neighborhood





3. Provide specific improvements to the network to resolve local delivery problems, such as closing the loop at the airport, upsizing pipes to the Island, and providing small PRV stations to neighborhoods in the Cliffside Zone.
4. Replace 2- to 6-inch diameter pipes with minimum 8-inch diameter pipes to resolve problems with fire flow capacity.

By far the largest change in the network, especially in the short term, will be to divide the main pressure zone into three zones. This will require a series of physical changes:

- Re-commission the existing PRV stations in order to re-establish the boundary between the Upper and Central Zones. Move the Center Street PRV station to a location farther east as part of dedicated line construction. Install new PRV stations along 10th West to establish the Lower Zone.
- Install new well pumps at all four wells. Each pump shall be equipped with a variable frequency drive motor, and the pumps will be sized to pump about 5 mgd to the Central Zone.
- Install new booster pumps at Center Street, Crockett and 6th East wells so these can boost to the Upper Zone in an emergency and to provide operational flexibility.
- Construct the east side well to provide backup supply to the Golf Course tanks for redundancy.
- Install new mains from Crockett and 6th East wells to connect these to the Central Zone. For Center Street, Crockett and the new east side wells, construct a dedicated transmission main to supply the Golf Course tanks in an emergency; also include a backup connection directly to the Upper Zone in an area on 6th East at 3rd North.
- Provide ground storage on the northeast side of the Central Zone, and elevated storage on the east side below the Island.

4.7 Year 2010 Hydraulic Analyses and Recommended Improvements

Development of Year 2010 System – Primarily Pressure Zoning

Table 2.1 shows that during the period 2005-2010, the population is anticipated to increase by 10.5% and employment by 19.3%. Most of these increases will occur in areas where the pipe network has already been established or is closely accessible.



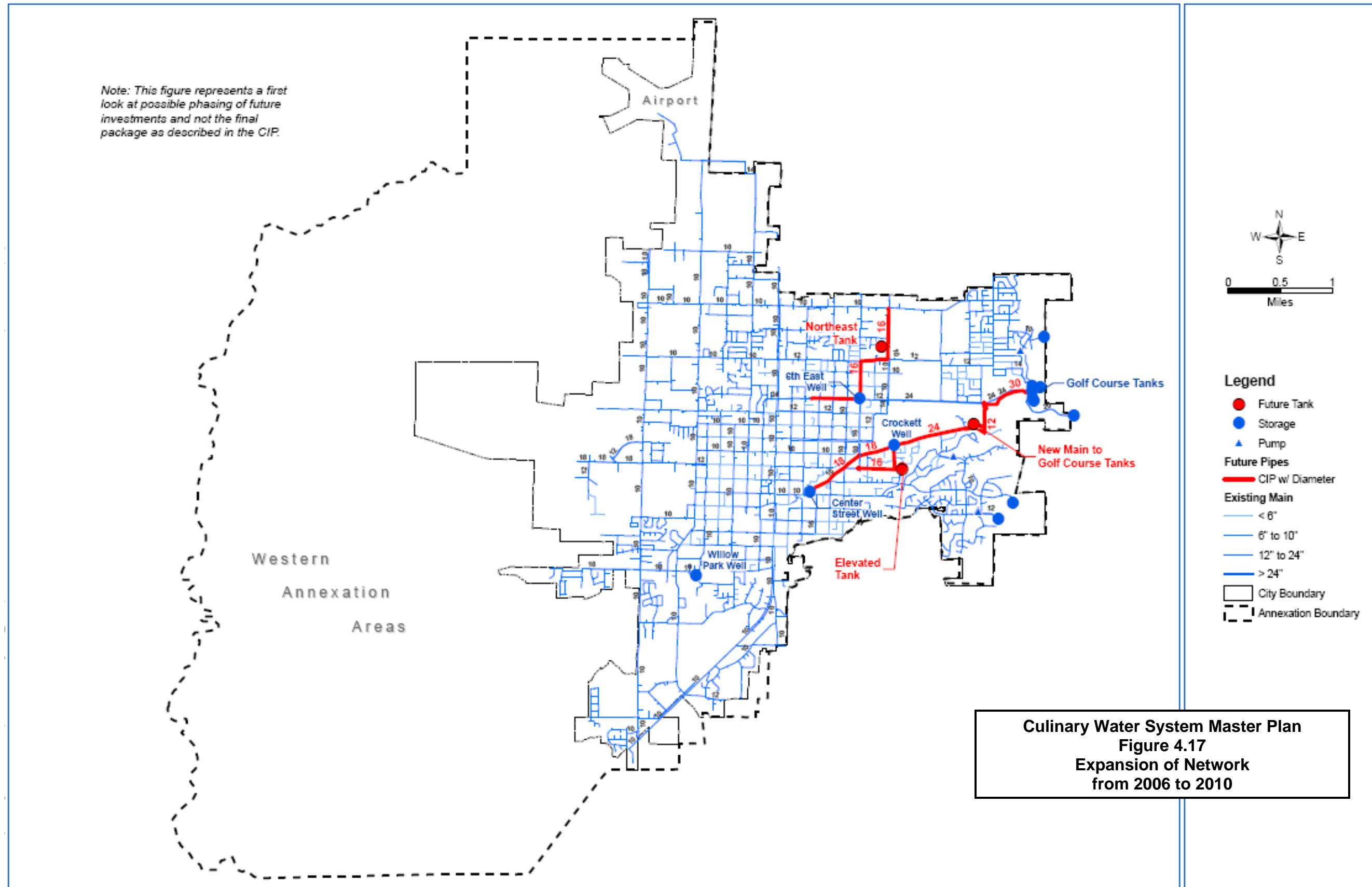
Considering the short planning horizon for 2010, development of the year 2010 hydraulic network must focus more on meeting overall operational objectives of the system and less on long-term expansion of the network to cover new areas.

Operational changes include dividing the main zone into three zones; making related transmission main, pump station and storage additions/modifications; and correcting limitations identified by modeling, such as at the airport or the Island.

Infrastructure which could wait until after 2010 includes:

- The new well on the east side.
- Elevated storage on the east side.
- Most piping on the west side of the city.
- The PRV stations along 10th West.

Figure 4.17 shows the year 2010 results, with new facilities highlighted in red, including new ground storage on the northeast side of the newly-established Central Zone, the dedicated transmission main from the Center Street well, and other transmission mains from the wells required to establish pressure zoning; also a closed loop at the airport, a larger supply line to the Island, and two small PRV stations in the Cliffside zone.





Year 2010 System Analyses – Scenarios

The City of Logan had identified three worst-case scenarios for year 2025 analyses, involving failure of two supply sources simultaneously on a maximum day. These scenarios were also used for year 2010 analyses:

1. DeWitt Springs pipeline **and** Golf Course tanks out of service.
2. Center Street well **and** 24" transmission main (above 6th East well) out of service.
3. 6th East **and** Crockett wells out of service.
4. Year 2010 peak hour.

The maximum day and peak hour runs for 2010 uses the same peaking factors as used in the 2025 scenarios. Fire flow analyses will not provide information different from what was obtained from the year 2025 analyses, and are not considered necessary.

DeWitt Springs Pipeline and Golf Course Tanks Out of Service

In this scenario, the City has turned on the booster pumps at Crockett and 6th East wells in order to maintain pressure in the Upper Zone. Crockett conveys water to a junction on the 24" transmission main downstream of the Golf Course tanks via the proposed dedicated transmission line.

The Center Street well is not turned on in the model for this scenario, but could be used to supply the Central Zone at a lower VFD setting.

In this scenario, supply to USU and Castle Hills has been interrupted. Unless USU has an emergency connection to the Upper Zone, or has sufficient capacity from its own well, it will not be able to maintain pressure in the USU zone. Castle Hills has no other connections to the network but GC5, and will therefore be completely without water once the 0.5 MG Castle Hills tank is drained down.

Table 4.9 shows the pressure results for this scenario, where the lowest pressures are generally near the storage tanks or at a high point on large transmission mains from the tanks.



Table 4.9
Year 2010 Maximum Day Model Results:
DeWitt Springs Pipeline and Golf Course Tanks Out of Service

Pressure Zone	High Press. (psi)	Low Press. (psi)	Comment
Upper Zone	121	10	low pressure adjacent to 24" main at high point near tanks and USU
Central Zone	95	44	
Lower Zone	102	89	Lower Zone is part of Central Zone in 2010, hence high pressures
USU Zones	54	<0	No pressure to this area
Castle Hills	110	48	Running off Castle Hills tank
Cliffside	120	28	low pressure near tank
Quail Bluffs	120	24	low pressure near tank and at one high point

Figure 4.18 shows the resulting pressures in the east/northeast part of the city. Note that without its own supply the USU area will not be able to maintain pressure with the GC5 tank out of service. Castle Hills still shows pressure because the Castle Hills tank is not yet drained in this steady state run. The rest of the network is handling the emergency fairly well, even with the Center Street well turned off.

Center Street Well and 24" Transmission Main Out of Service

This scenario primarily looks at impacts on the Central Zone, where the Center Street well is out of service, and supply from the Upper Zone via the 24" main is compromised immediately upstream of the 6th East well. All the other wells and tanks are functioning.

Table 4.10 shows the results from this scenario run. The network is able to handle the supply restrictions quite well because water from the Upper Zone can spread to laterals and then travel down to the Central Zone via other mains to the PRV stations.

Pressures are generally higher than in the corresponding 2025 scenario because demands are lower but the pump settings have not been changed. In reality the well pumps would be regulated by VFDs and therefore be able to operate at lower head based on pressure signals from select points in each pressure zone.

Pressures are also higher in the Lower Zone because this zone is not yet established. Therefore the full pressure from the Central Zone leads to excess pressures here.

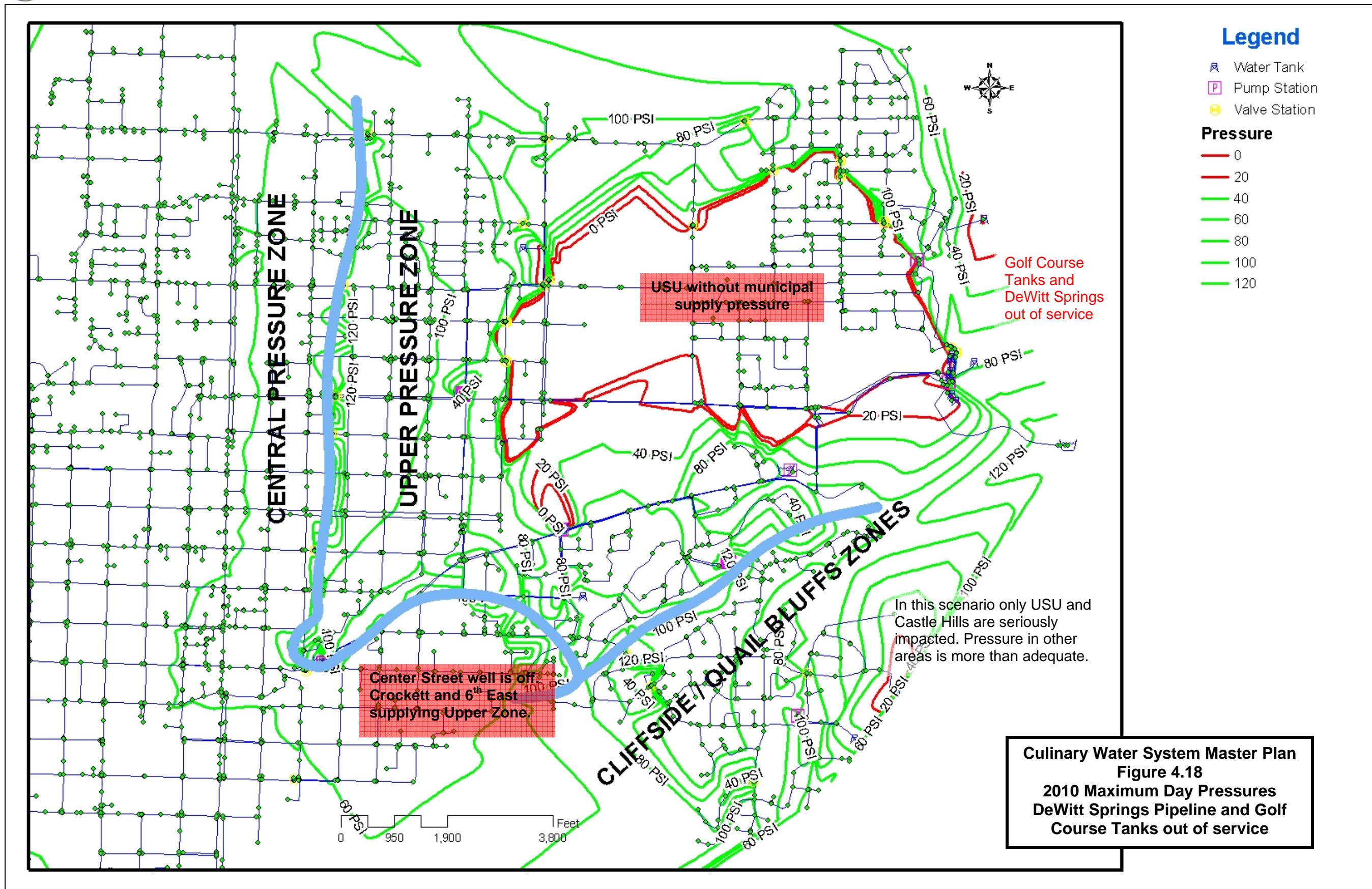




Table 4.10
Year 2025 Maximum Day Model Results:
Center Street Well and 24" Transmission Main Out of Service

Pressure Zone	High Press. (psi)	Low Press. (psi)	Comment
Upper Zone	133	24	low pressure adjacent to 24" main at high point near tanks
Central Zone	93	48	
Lower Zone	106	92	Lower Zone is part of Central Zone in 2010, hence high pressures
USU Zones	144	73	
Castle Hills	123	51	
Cliffside	120	28	low pressure near tank
Quail Bluffs	120	24	low pressure near tank and at one high point

6th East Well and Crockett Well Out of Service

This scenario looks at the consequences of having two supply sources out of service in the Central Zone. Table 4.11 shows the results for this scenario, which show the same high pressures as the previous scenario and no significant change in overall pressures.

Table 4.11
Year 2010 Maximum Day Model Results:
6th Street and Crockett Wells Out of Service

Pressure Zone	High Press. (psi)	Low Press. (psi)	Comment
Upper Zone	134	24	low pressure adjacent to 24" main at high point near tanks
Central Zone	99	48	
Lower Zone	107	92	Lower Zone is part of Central Zone in 2010, hence high pressures
USU Zones	144	73	
Castle Hills	123	51	
Cliffside	120	28	low pressure near tank
Quail Bluffs	120	24	low pressure near tank and at one high point



Peak Hour Analysis

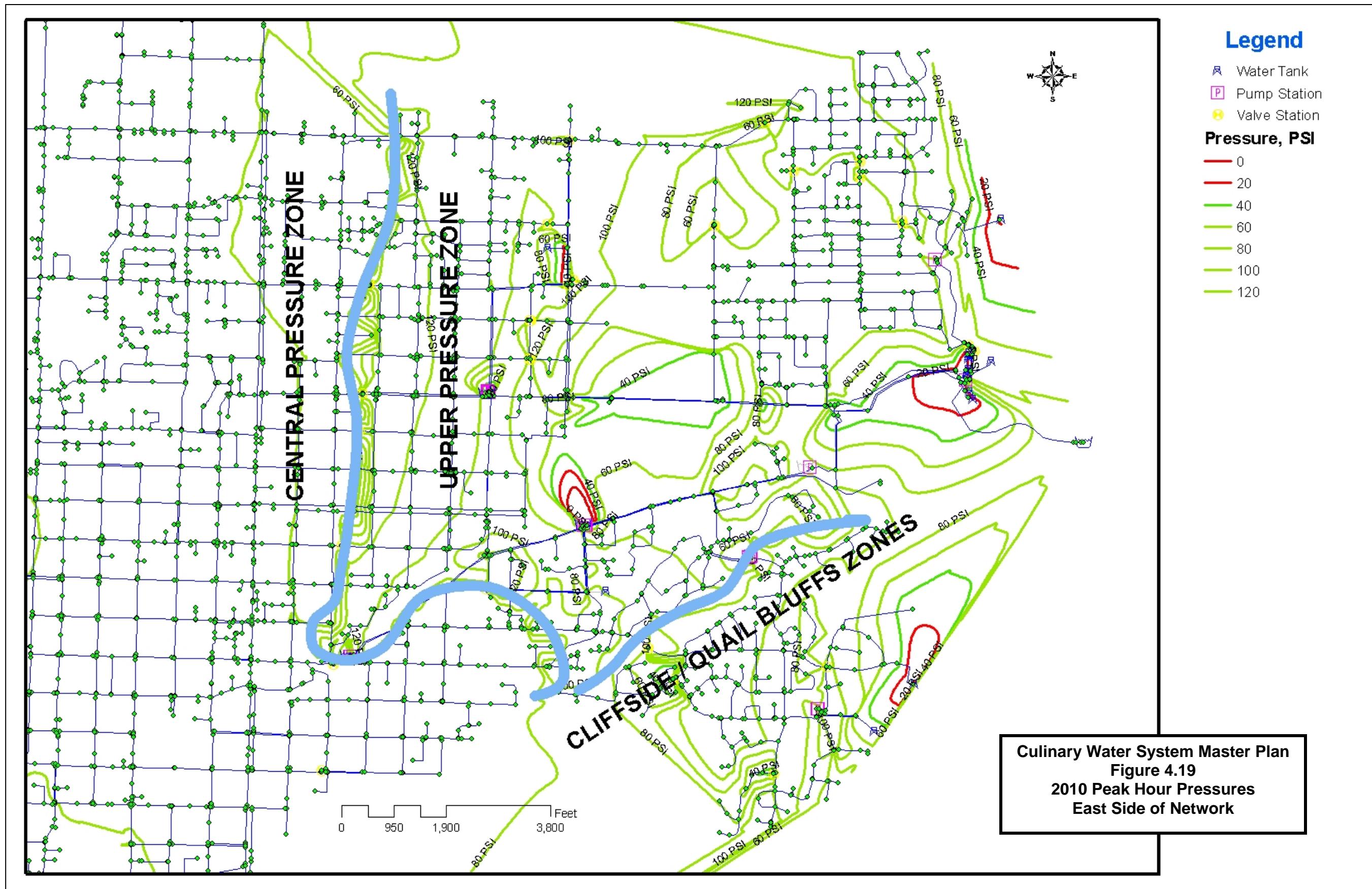
The year 2010 peak hour analysis assumes a peak hour to average day factor of 2.4; and all wells, tanks and springs are available to supply peak hour demands of 27 mgd.

The peak hour demands result across the board in lower peak pressures in the pressure zones, but these are yet higher than the 2025 peak hour pressures because the pumps are meeting a lower demand.

Figure 4.19 shows the east side of the network, while Table 4.12 summarizes and comments on the results from this modeling run. With the exception of a small area adjacent to the tanks, all of the red pressure contours in this figure represent **high** pressures in the network.

Table 4.12
Year 2025 Peak Hour Model Results

Pressure Zone	High Press. (psi)	Low Press. (psi)	Comment
Upper Zone	132	24	low pressure at high points near tanks, 24" main, and high point at closed USU valve
Central Zone	96	44	
Lower Zone	104	86	Lower Zone is part of Central Zone in 2010, hence high pressures
USU Zones	134	68	
Castle Hills	122	51	
Cliffside	120	28	low pressure near tank
Quail Bluffs	119	24	low pressure near tank and at one high point





Year 2010 Summary of Recommended Improvements

The year 2010 analyses confirm that the most urgent improvements consist in dividing the main pressure zone into at least two pressure zones.

Recommended Improvements:

1. Divide the main pressure zone into the Upper Zone and Central Zone. This will require changes to network storage, pumping, transmission and pressure control.
2. Provide specific improvements to the network to resolve local delivery problems, such as closing the loop at the airport, upsizing pipes to the Island, and providing small PRV stations to neighborhoods in the Cliffside Zone.
3. Replace 2- to 6-inch diameter pipes with minimum 8-inch diameter pipes to resolve problems with fire flow capacity.



5.0 GIS Review

The purpose of the GIS review is to identify additional features and applications which can make the GIS more useful to the City of Logan and identify options for better integration of GIS and hydraulic models. Actual implementation of these options were planned to occur outside the timeframe of the Water Master Plan study.

However, during the Water Master Plan study, the City agreed in August 2005 to proceed with the integration of GIS and hydraulic models. This required the City to update the water distribution network in GIS, before Black & Veatch would proceed to model the water network based on the updated GIS data. The City also decided to apportion future population projections by Traffic Area Zones (TAZ) using GIS.

The intent was to pilot this approach for the water network, and later base wastewater collection system planning on GIS as well.

5.1 Description of the City GIS

The City GIS Department maintains geographical information for various municipal purposes. ArcGIS version 9.1 is used to maintain a geographical database for streets, water, sewer, electrical and other services. The City GIS system also includes municipal boundary, population, land use, buildings, living units, elevation contours, aerial photography, taxable property boundaries and other information for use by various City departments. ArcGIS is an industry standard tool, with a large user base and well-supported software.

GIS Department staff is well-trained and conversant in the use of ArcGIS. The department has played a key role in further developing information useful for City community development and population projections that have served as an information basis for the Water Master Plan activities.

Having a well-maintained GIS will allow the City to maintain an accurate geographical database of the water distribution network. This can become a tool for asset management and pipe replacement planning. It can serve as a practical tool for guiding and documenting operations and maintenance, and can be used by the Engineering Department in planning expansion and improvement of the water distribution system.



The City of Logan recently purchased CityWorks, a GIS-based asset maintenance management system that can be used for service requests and maintenance planning. The timely conversion of water network information to GIS files will facilitate utilization of CityWorks, which will in turn help in implementing the Water Master Plan CIP.

5.2 Updating GIS Network Data

When the City made the decision to pilot the Water Master Plan using GIS files, review of these files revealed that a number of inadequacies needed correcting before the files could be used to develop hydraulic models.

The City began updating the water GIS data in September 2005 and continued into early 2006. Pipe failure data was incorporated over the subsequent months. This intensive effort required cooperation between the GIS Department and the Water-Wastewater Division to verify network information.

Network coordinates for pipe and valve locations were obtained using global positioning system (GPS) field measurements. However, some gaps remained in the data generated, so there is still room for improving documentation procedures, including topology (“snapping” pipe endpoints to each other and to valves).

Also, City staff researched the drawings archive and carried out field verifications in order to clarify many network issues, while the GIS Department manually-entered pipe failure data that would be necessary to Pipe Replacement Planning. This was an iterative process involving the GIS Department, Water-Wastewater Division, and Black & Veatch.

Upon completion of the GIS data cleansing, and after incorporation into the hydraulic network modeling program, many connectivity and other issues had yet to be resolved in order to create a functioning hydraulic model that could then be used for existing system analyses.

In the interests of time, hundreds of connectivity issues were resolved in H2OMAP using internal tools for this purpose – which means that these errors still need to be resolved in the GIS files.



5.3 GIS Network Data not currently Available

For the hydraulic model, water demands were based on population and employment distribution in Traffic Area Zones (TAZ) established by regional planners. Unfortunately, TAZ on the west side of town are quite large – which means that they may not adequately identify current or projected population distribution.

The Community Development Department prepared sketches showing the location and phasing of future developments. These plans were used as the basis for future population distribution, and then related back to the TAZ for allocation to existing and future nodes.

Water demands are correlated to the TAZ-based population distribution under the assumption that per capita consumption is the same across the city. However, it can be expected that upper-income homes with pools and large irrigated lawns will have a higher per capita consumption than high-density housing or student dormitories, showing the limitation of this approach.

The use of TAZ-based population distribution, assigning this to nodes, and allocating demand on a per capita basis is standard procedure. However, a better approach may be possible in the future once more accurate data is available:

A better approach would be to link water billing data from the Customer Information System to the GIS database. Then metered consumption can be tied to the nearest demand node in the water distribution network. This would allow more accurate distribution of water demands in the hydraulic model, and evaluation of seasonal and even monthly demand patterns.

This would require linking the customer meter address or (ideally) GPS coordinate to the parcel. Though the meter is usually in the public right-of-way, this GIS correction could be performed using an automated proximity relation.

Black & Veatch recommends the City consider taking this step that will also facilitate sewer master planning by allowing better estimating of sewer flows throughout the city. Black & Veatch can support the City in this task should the City need assistance.



5.4 GIS and Hydraulic Model Integration

Hydraulic models are rapidly becoming more integrated with GIS, and this trend can be expected to continue in the future. This is leading to more frequent use of all-pipes models, such as the one developed for the City of Logan. The underlying key is that GIS data must be accurate and in a form readily usable by the hydraulic model.

At the same time, GIS is increasingly playing a central role in other mission-critical needs, such as integration with other databases and systems. Strategic database design may be necessary to strengthen the useful role of GIS, so that databases can better communicate and data only needs to be maintained once.

Review of Key Modeling Programs

There are several water distribution modeling programs available, and these are being continually improved. Each of these programs includes steady state hydraulic modeling, water quality modeling, and fire flow analysis as standard features, and all are based on the EPANET engine. Programs include:

- Wallingford Software: InfoWorks WS, InfoNet
- MWH Soft products: InfoWater, H2OMAP and H2ONET
- Bentley: WaterGEMS

Wallingford Software products include InfoWorks WS for Water Supply, and InfoNet. InfoWorks includes a self-calibration module. More importantly to the City of Logan, it has seamless import of ArcGIS files. It also has asset management, water hammer modeling and other functions. InfoNet software provides a number of data and network cleansing tools useful for creating/updating the model network, and can be used to import survey databases.

MWH Soft products include InfoWater, H2OMAP, and H2ONET. The Logan Water Master Plan has been developed in H2OMAP, which requires importing ArcGIS files into the modeling program. InfoWater, the newest product, has the benefit of seamless use of ArcGIS files directly in the program. H2OMAP runs in an environment similar to ArcGIS, while H2ONET runs with AutoCAD. The suite version of these programs includes demand allocator and other support modules.



Bentley Systems products include WaterGEMS, a system with licenses for four different platforms – ArcGIS, AutoCAD, MicroStation and as a stand alone system. This means that users can share the same data across different platforms if they have the appropriate Bentley licenses. The newest version (August 2006) includes additional features such as the ability to model variable speed pumps, pressure-dependent demands, etc. Water Hammer is separate. Bentley has purchased WaterCAD and appears to be phasing this out in favor of WaterGEMS.

Selection of Hydraulic Modeling Program

Among all three systems, integration with GIS is becoming the standard. The selection of a system is often based on individual client preferences – such as familiarity with the product, ease of operation, platforms being used by the city, etc. And promotional prices for new products are quite common.

MWH Soft products probably have widest use in the U.S. The model for the City of Logan's Water Master Plan was prepared using H2OMAP.

The published retail price of MWHSoft products is less than other industry used water distribution system modeling products. InfoWater and H2OMAP cost \$11,000 for an 8000-links program, and another \$1,000 for add-in modules. The 8000-links program may be relevant if the City desires to run the model with all hydrants – with about 1,400 pipe segments listed as Hydrant, the total number of links in the City's model would be about 7,200 links.

Black & Veatch recommends that the City continue with H2OMAP or convert to InfoWater for the following reasons:

- The present model was developed in H2OMAP. Therefore, the City does not have to pay again for model development and model migration. The model is set up, calibrated and ready-to-run.
- The present model can be uploaded directly into InfoWater. However, for future updates connectivity and other issues should be resolved in the GIS database.
- The cost of both programs is equal and competitive with similar products.



5.5 Use and Maintenance of GIS Network Data

Ideally, the GIS can be integrated with other databases and systems. The strength of this approach is that data can be more easily accessible for a wider range of uses, and that data only has to be updated once in one location.

The City GIS could be integrated with:

- The Customer Information System, including customer billing information. However, the current system is the AS400, which may not be amenable to integration.
- Work Order Management System. The City has accomplished this.
- Asset Management System. The City purchased CityWorks in 2006 and has now accomplished this.

For water supply, the key to successful use of GIS is to make GIS applications accessible, and make available and maintain GIS data.

Use of GIS Network Data

The GIS has already become an invaluable tool for network maintenance by providing printable maps – complete with network coordinates and aerial background. Operations staff can readily locate pipes and valves for maintenance repair or service. They can view the GIS data, but do not have administrative rights to alter the information.

Further benefits could be realized by including additional information in the GIS. For example, type of pipe or valve failure could be specified and recorded. Failures might include leakage from valve stem packing, failed valve stems, loose pipe joints, cracking of pipe joints, cracking along pipe length, or corrosion holes. This detailed information would help identify trends/patterns and adjust pipe replacement planning, just as the City previously evaluated fire hydrant performance and obtained valuable insight.

As more types of data become available, the GIS will prove to be a powerful decision-support tool serving a variety of mission-critical applications.

For water distribution network modeling, updates to the GIS database, such as new pipes, valves, pumps and tanks, can be imported into the model of the existing system. However, care must be exercised to not compromise network elements already imported into the model.



Maintenance of GIS Network Data

The City has made important progress by updating and correcting the water network and by providing data access to the operations and maintenance staff. However, after the enormous initial effort to update and verify the water network, focus must shift to **maintaining** this vital information. This requires defined expectations, established procedures and user-friendly tools.

Black & Veatch has discussed these data maintenance issues with the GIS Department. The Department arrived at the same conclusions and had already begun implementing procedures in August 2006 that correspond to B&V's recommendations.

For repairs or installations:

- (1) The Water-Wastewater Division will submit a brief request form to the GIS Department. Work crews can use the GIS Viewer application to print a map showing the pipe or valve to be repaired or installed. Upon work completion, the updated map can be submitted, along with the completed request form.
- (2) The GIS Department then assigns a tag ID to this work request. This tag ID will be used to track the work.
- (3) The Division then locates the repair or installation using GPS survey equipment. This information is returned to the GIS Department for entry into the GIS. We suggest that the GIS Department and Water-Wastewater Division establish a standard form to collect this data. This form could be modified from the form currently used to document work in the network.
- (4) The GIS Department uses the tag ID and GPS coordinates to validate and enter the repair or installation information into the GIS. This could be done on a periodic-basis or as-need basis. The form could be returned to the Division for hard-copy archiving.

This procedure requires close collaboration between the Water-Wastewater Division (performing and documenting the field work), and the GIS Department (tagging and recording this work).

Another option is to use the latest technology employing Field Data Collector equipment – commercially-available hand-held field data systems like the ESRI ArcPAD. This approach in essence uses technology to facilitate the procedures described above. Such systems are becoming increasingly user-friendly, which is important for field work.



Such mobile GIS equipment would allow the repair crew to view the GIS map of the network, verify in the field what they see on the map, fill out electronic forms in the field, incorporate GPS measurements (using GPS equipment with survey-grade accuracy), and make corrections and updates to the database information. They may find that the diameter, pipe material or location is incorrect and can correct this immediately in the field, as well as record repairs, replacements, and new installation. This information is stored in a temporary database.

After field tasks are completed, the changes to GIS information can be downloaded, and then the GIS Department would validate these changes before updating the actual GIS files. Black & Veatch recommends that the City consider this approach, which will help avoid paperwork and facilitate GIS maintenance and updating procedures, and focus on systems that offer survey-grade accuracy, are user-friendly, and compatible with mainstream GIS software.

5.6 Corrosion Attribute Assignment

The USDA Natural Resources Conservation Service (NRCS) maintains GIS-based information on soil corrosion potential, and provides a corrosivity rating for steel and concrete pipe, with corrosivity potential designations of High, Moderate and Low. The City GIS Department incorporated the publicly-available NRCS files to the GIS and developed a routine for assigning corrosivity categories to the pipe data.

Using the NRCS files, Black & Veatch developed a soil corrosion potential rating system applicable to cast iron and ductile iron pipe. These ratings were also incorporated in the Pipe Replacement Planning.

Soil Corrosion Potential

Soil corrosion potential with respect to cast iron, ductile iron, and steel pipe is based on *External Corrosion – Introduction to Chemistry and Control* (AWWA M27, 1987), Table 3-1 (Soil-Test Evaluation for Gray or Ductile Cast-Iron Pipe) and Table 3-3 (Relation of Soil Corrosion to Soil Resistivity). AWWA Table 3-1 indicates that 5 factors impact soil induced corrosion of cast iron and ductile iron pipe:

- Resistivity
- pH
- Redox potential
- Sulfides



- Moisture

NRCS Chemical Soil Properties Tables were used to determine soil resistivity (from salinity) and pH. NRCS Water Feature Tables were used to determine depth to groundwater (month of June). Using this NRCS data, soil corrosion potential for cast iron and ductile iron pipe, based on 3 of the 5 soil characteristics listed in AWWA M27 (Table 3-1), was determined for soils found in the Logan area. Soil corrosion potential (“score” based on the above factors) for cast and ductile iron pipe is given in Table 5-1.

AWWA M27 (Table 3-1) indicates that soils with a soil corrosion potential “score” of 10 or higher are considered corrosive with respect to cast iron and ductile iron pipe. Soils in the Logan area have cast iron and ductile iron corrosivity scores ranging from 8 to 13 (Table 5-1).

Statistical analyses of leak data were used to determine the percentage of pipe leaks to the percentage of pipe length for the Logan area soil types. Seven soil types were identified with disproportionately high leak rates. The soil corrosion potential “scores” for these soil types are summarized below:

Soil Type and Cast Iron and Ductile Iron Pipe Corrosivity Score:

- Am: 8
- GsA: 13
- GsB: 10
- NcA: 11
- Sg: 13
- SvA: 8
- TmC: 8



Table 5-1
Soil Corrosion Potential – Cast Iron, Ductile Iron, and Steel Pipe

Soil Symbol (musym)	Soil Name	Chemical Properties		Physical Property, Water Table Depth (ft)	Cast & Ductile Iron Pipe Corrosivity Score				Steel Pipe Corrosivity Rating	
		pH	Salinity (mmhos/cm)		Resist- ivity	pH	Moisture	Total	Description	Score
Ak	Airport Silty Clay Loam	8.50	18.00	2.5	10	3	1	14	Bad	10
Am	Airport-Salt Lake Complex	8.50	18.00	1	10	3	1	14	Bad	10
Cd	Cardon Silty Clay	8.20	1.00	3	8	0	1	9	Bad	10
Ck	Collett Silty Clay Loam	7.90	2.00	2.25	10	0	1	11	Bad	10
Gp	Gravel Pit	NA	NA	NA	NA		NA	10	NA	NA
GsA	Greenson Loam (0-3 %)	8.50	6.00	3.75	10	3	1	14	Bad	10
GsB	Greenson Loam (3-6 %)	8.45	6.00	3.75	10	0	1	11	Bad	10
HgE2	Hillfield-Timpanogos Silt Loams	8.20	2.00	NA	10	0	NA	10	Bad	10
HhE2	Hillfield-Timpanogos Silt Loams	8.20	2.00	NA	10	0	NA	10	Bad	10
Lr	Logan Silty Clay Loam	8.20	2.00	1.75	10	0	1	11	Bad	10
MIA	Millville Silt Loam	8.20	1.00	3.25	8	0	1	9	Bad	10
NcA	Nibley Silty Clay Loam (0-3 %)	8.50	1.00	3.25	8	3	1	12	Bad	10
PIA	Parlo Silt Loam (0-3 %)	7.60	1.00	NA	8	0	NA	8	Bad	10
PIB	Parlo Silt Loam (3-6 %)	7.60	1.00	NA	8	0	NA	8	Bad	10
Pn	Payson Silt Loam	9.10	20.00	3.25	10	3	1	14	Bad	10
Pu	Provo Loam	7.60	0.00	3.6	0	0	1	1	Excellent	0
Pv	Provo Gravelly Loam	7.60	0.00	2.5	0	0	1	1	Excellent	0
RCG2	Richmond Very Stony Loam	7.90	1.00	NA	8	0	NA	8	Bad	10
RhA	Ricks Gravelly Loam (0-3 %)	6.50	1.00	NA	8	0	NA	8	Bad	10
RhB	Ricks Gravelly Loam (3-6 %)	6.50	1.00	NA	8	0	NA	8	Bad	10
RhC	Ricks Gravelly Loam (6-10 %)	6.50	1.00	NA	8	0	NA	8	Bad	10
Rs	Roshe Springs Silt Loam	8.50	2.00	2	10	3	1	14	Bad	10
Rt	Rough Broken Land	NA	NA	NA	NA		NA	10	NA	NA
Se	Salt Lake Silty Clay	8.50	3.00	1.25	10	3	1	14	Bad	10
Sg	Salt Lake-Roshe Springs Complex	8.50	3.00	1.25	10	3	1	14	Bad	10
SvA	Steed Gravelly Loam	7.90	1.00	4	8	0	1	9	Bad	10



Table 5-1 (Cont.)
Soil Corrosion Potential – Cast Iron, Ductile Iron, and Steel Pipe

Soil Symbol (musym)	Soil Name	Chemical Properties		Physical Property, Water Table Depth (ft)	Cast & Ductile Iron Pipe Corrosivity Score				Steel Pipe Corrosivity Rating	
		pH	Salinity (mmhos/cm)		Resist- ivity	pH	Moisture	Total	Description	Score
SvC	Sterling Gravelly Loam (6-10 %)	7.90	1.00	4	8	0	1	9	Bad	10
SwC	Sterling Gravelly Loam (6-10 %)	8.20	1.00	NA	8	0	NA	8	Bad	10
SwD	Sterling Gravelly Loam (10-20)	8.20	1.00	NA	8	0	NA	8	Bad	10
SwF2	Sterling Gravelly Loam (20-50)	8.20	1.00	NA	8	0	NA	8	Bad	10
TmA	Timpanogos Silt Loam (0-3 %)	8.20	1.00	NA	8	0	NA	8	Bad	10
TmB	Timpanogos Silt Loam (3-6 %)	8.20	1.00	NA	8	0	NA	8	Bad	10
TmC	Timpanogos Silt Loam (6-10 %)	8.20	1.00	NA	8	0	NA	8	Bad	10
TnA	Timpanogos Silt Loam, Deep Water	8.20	1.00	4	8	0	1	9	Bad	10
W	Water	NA	NA	NA	NA		NA	NA	NA	NA
Wn	Winn Silt Loam	8.20	1.00	3	8	0	1	9	Bad	10
Wp	Winn-Provo Complex	8.20	1.00	3	8	0	1	9	Bad	10

Notes:

Source: NRCS Chemical Soil Properties and Water Features Tables

Water Table Depth is June Upper Limit

Cast Iron, Ductile Iron, Steel Pipe Corrosivity Potential from *External Corrosion – Introduction to Chemistry and Control* (AWWA M27, 1987)



The soil corrosion potential “scores” for the 7 soil types above indicate that 3 of the 7 soil types have corrosion potential scores of 8. The general conclusion drawn from the above finding is that soil types with corrosion potential scores of 8 and above should be considered corrosive with regards to gray iron pipe. This includes nearly all the soil types in the Logan area.

AWWA M27 indicates that soil chemical and physical properties, moisture content, and stray electrical currents can impact soil corrosion potential with respect to steel pipe. AWWA M27 goes on to say that soil resistivity is generally the most important factor with respect to evaluating corrosion potential of steel pipe. AWWA M27 Table 2-7 gives descriptive ratings for soil corrosion potential for steel pipe based on soil resistivity. The information given in AWWA M27 Table 2-7 was combined with soil resistivity data to develop soil corrosion potential descriptions and “scores” with respect to steel pipe (see Table 5-1). All soil types in the Logan area (with resistivity data) are rated as “Bad” with respect to corrosion potential for steel pipe.

Conclusion and Recommendations

In summary, the above evaluation indicates that the soil corrosion potential for nearly all the soil types found in the Logan area are considered high, with respect to cast iron, ductile iron, and steel pipe – meaning that in practice other factors such as depth to water table, pipe age and pipe material may play a greater role in pipe survival.

It is recommended that the Logan GIS “Soils” layer/coverage be updated to include Fields for the five soil parameters that impact corrosion of cast iron and ductile iron pipe (resistivity, pH, redox potential, sulfides, and moisture).

In addition, a data Field representing the corrosion potential (score), with respect to cast iron and ductile iron pipe, should be added to the GIS Soils layer. Field soils testing for the above parameters is recommended during pipe leak repairs to develop a more accurate and detailed soil properties database and improve future estimates of soil corrosion potential within the City.



6.0 Pipe Replacement Plan

6.1 Pipe Replacement Methodology

Pipe replacement planning has as its primary objective development of annual pipe replacement budgets. Traditionally this was accomplished by a qualitative approach that looked at overall indicators for pipe age, pipe failure data, and other criteria. The level of detail – and accuracy – was limited by the level and form of information available.

The City of Logan has converted the entire water network to GIS database files. Later, pipe failure data was also added to the GIS. While this required considerable effort by the GIS Department and the Water-Wastewater Division, it has significant benefits not only for operations and engineering functions, but specifically for pipe replacement planning.

GIS-based pipe replacement planning can be more accurate in preparing annual pipe replacement budgets, but also has the advantage that it can assign replacement priorities *to every pipe in the network*. This tool makes it possible to identify packages of pipes falling into selected prioritization ranges, which allows for better scheduling and budgeting of pipe replacement.

The methodology used for pipe replacement planning is to:

- Develop pipe replacement prioritization categories.
- Develop pipe replacement prioritization scores and weights.
- Assign scores and weights for each prioritization category to network piping.
- Develop water main survival curves and assign unit replacement costs.
- Integrate prioritization with replacement costs to develop budget options.

There are two products from the pipe replacement planning:

1. Pipe prioritization scores.
2. Annual pipe replacement budgets.



6.2 Pipe Prioritization Categories and Scores

Pipe Prioritization Categories

Pipe prioritization takes into account those factors, typically age, corrosivity or failure rates, which would indicate that pipe is approaching the end of its useful lifetime. The end of useful lifetime is the point in time at which the cost to maintain the pipe approaches the cost to replace it, but there are other factors which could bring the decision to replace pipe forward in time, and these are also prioritization categories.

Pipe prioritization categories were selected in consultation with City staff on the 19th of May 2006. The categories selected are:

- Pipe age
- Pipe leaks/mile
- Pipe service pressure
- Soil corrosion potential
- Pipe criticality
- Coordination with other projects
- Supervisor input

Pipe Prioritization Scores

Scores were developed for each pipe prioritization category, where the highest scores are those pipes receiving highest priority within that category.

Pipe age is one useful indicator of remaining pipe lifetime. For the evaluation model, pipe 75 or more years old was given a score of 3, pipe 50 years or older a score of 2, and pipe 25 or more years old a score of 1. Pipe less than 25 years old was given a score of 0.

Pipe leaks/mile is another indicator of remaining pipe lifetime. The City of Logan has the experience of repairing a pipe, only to find several previous repairs within just a few feet of the new failure location. In such cases it is more cost-effective to replace the entire pipe segment.

As an example, the frequent repairs along Main Street are not only expensive, but disrupt local business and result in customer complaints. Once pipe failure information is maintained in the GIS, City staff will be able to track repairs and can better decide when to replace a main instead of just conducting spot repairs.



For the evaluation model, pipe with three or more failures per mile was given a score of 3, pipe with one to three failures was given a score of 2, pipe with less than this was given a score of 1, and pipe with no recorded failures was given a score of 0.

Pipe service pressure in the City of Logan was also chosen as a prioritization category. This is because pressure in some areas exceeds 200 psi, resulting in water hammer and lower pipe lifetimes for all types of pipe.

For the evaluation model, pipe with static service pressure of 150 psi or higher was given a score of 2, pipe with a pressure of 100 psi to 150 psi a score of 1, and pipe with pressure less than 100 psi a score of 0.

Corrosion potential was chosen as a prioritization category due to external corrosion failures both on the DeWitt Springs pipeline and in ductile iron pipe on the west side of town. **Internal** corrosion, common in many networks, is not reported to be a problem in Logan, and the Regulatory Review analysis of water quality data supports this conclusion. Therefore internal corrosion was not chosen as a prioritization category. However, GIS database files from USDA NRCS showed that most soils in the Logan service area are moderate to highly corrosive, so **external** corrosion appears to be a valid concern.

Soil corrosion potential was determined based on analysis of USDA NRCS Chemical Soil Properties Tables, and relating this information to AWWA M27 (manual on external corrosion). Salinity data reported by NRCS was converted to resistivity for comparison with the ten-point system employed in Table 3-1 in AWWA M27.

Resistivity, depth to groundwater, and pH were used to develop total corrosion potential points for each type of soil in the Logan service area as they relate to steel and cast iron/ductile iron pipe. For steel pipe, soils were consistently rated at 10 points, while for cast iron/ductile iron pipe, values ranged from 1 to 14. AWWA M27 Table 3-1 indicates that for soils with total points of 10 or higher, the soil is corrosive.



Pipe leak rates were compared in GIS to NRCS soil types in order to identify correlations. The weakness with this approach is that current leak history does not include the *type* of leak – which could be corrosion pitting, but could also owe to sheared pipe or joint failure, which are not related to soil corrosion. The City now documents each failure so that a better base of information will be available for future use.

However, under the assumption that leaks will proportionately be higher in corrosive soils, seven soil types were identified with disproportionately high leak rates. These were rated at 8 to 13 points in the NRCS rating system. The analysis concludes that soil types with corrosion potential of 8 points or more should be considered highly corrosive for cast iron/ductile iron pipe. This includes nearly all soil types in the Logan service area.

For the evaluation model, pipe in soils with corrosion potential of 8 points or more was given a score of 2, in soils with corrosion potential of 4 to 8 points a score of 1, and in soils with corrosion potential of less than 4 points a score of 0. Since nearly all soil types in the Logan service area are highly corrosive, almost all the pipes are given a score of 2.

Pipe criticality was based on two criteria – mains serving critical facilities, such as pump stations, wells and tanks; and mains supplying critical customer services. All pipes to critical water supply facilities were given a high rating, while secondary lines to these facilities were given a medium rating.

Pipes serving medical care facilities and assisted living center were given a high rating. Elementary schools, which are designated emergency shelters, and the USU campus, were also given a high rating. Large churches, middle schools and the high school, which might serve as secondary shelters, were given a medium rating.

For the evaluation model, pipe with high criticality was given a score of 2, pipe with medium criticality a score of 1, and pipe with low/no criticality a score of 0. It is worth mentioning that pipes with the highest criticality represented only 11.4% of total network length and those with medium criticality represented 3.0% of total length.

Coordination with other projects is a prioritization category that allows bringing pipe replacement forward in time if, for example, a street is going to be widened or other utilities installed or replaced, in which case it would be more economical or convenient to replace this pipe at the same time.



City of Logan staff identified and assigned pipe segment ratings for this category. Pipes rated high were given a score of 5, medium-high a score of 3, medium a score of 2, and pipes not rated were given a score of 0.

Pipes rated high in this category represented only 2.4% of total network length, medium-high only 2.3% of total length, and medium only 2.0% of total length.

Supervisor input is a prioritization category that allows bringing pipe replacement forward in time in conjunction with overall development needs, emergency services, future plans and other considerations that are best known by City staff. It also allows the City to better package pipe replacement investments. For example, if four blocks in a five-block section are given highest replacement priority, but the middle block is scored slightly lower, the City can raise the priority of the middle block to make a convenient cost-effective package.

City of Logan staff identified key pipe replacement projects and assigned ratings to pipe segments. Pipes rated high were given a score of 10, medium-high a score of 5, medium a score of 3, and those with no rating were given a score of 0.

Pipes rated high in this category represented only 1.4% of total network length, medium-high only 0.4% of total length, and medium only 1.4% of total length. What this low percentage and that for coordination with other projects means is that City staff has been careful and selective in prioritizing pipes.

6.3 Pipe Replacement Analysis and Prioritization

The intent of pipe replacement planning is to develop annual pipe replacement budgets based on a logical prioritization of pipe replacement needs.

Utilities that follow a well-developed approach replace pipes with highest priority first. The net effect is that the utility focuses first on those pipes whose replacement will give the greatest benefit for the investment in terms of improving the level of service and decreasing maintenance costs.



Incorporating Pipe Prioritization Categories for Pipe Age, Leaks/Mile and Pressure

As part of analysis, B&V analyzed patterns within the specific categories of pipe age, leak rates, and pressure. These are shown in Figures 6.1, 6.2 and 6.3, respectively.

Figure 6.1 shows pipes by age. The oldest 10% of the network are shown in red. These pipes are 59 to 136 years old. This figure illustrates the preponderance of older pipes in the central section of the city, and also areas adjacent to USU. Note that older pipes tend to form longer continuous pipe segments, which would make it easier to group pipe segments into practical replacement projects.

Figure 6.2 shows pipes by leaks per mile per year. Those segments with three or more leaks per mile per year are shown in red, and represent about 10% of the total network. Here there are fewer continuous pipe segments.

Comparing with the previous figure, it is noteworthy that some of the newest pipes are already experiencing leaks, for example in the area of 6th North and 6th West (Palatial Living).

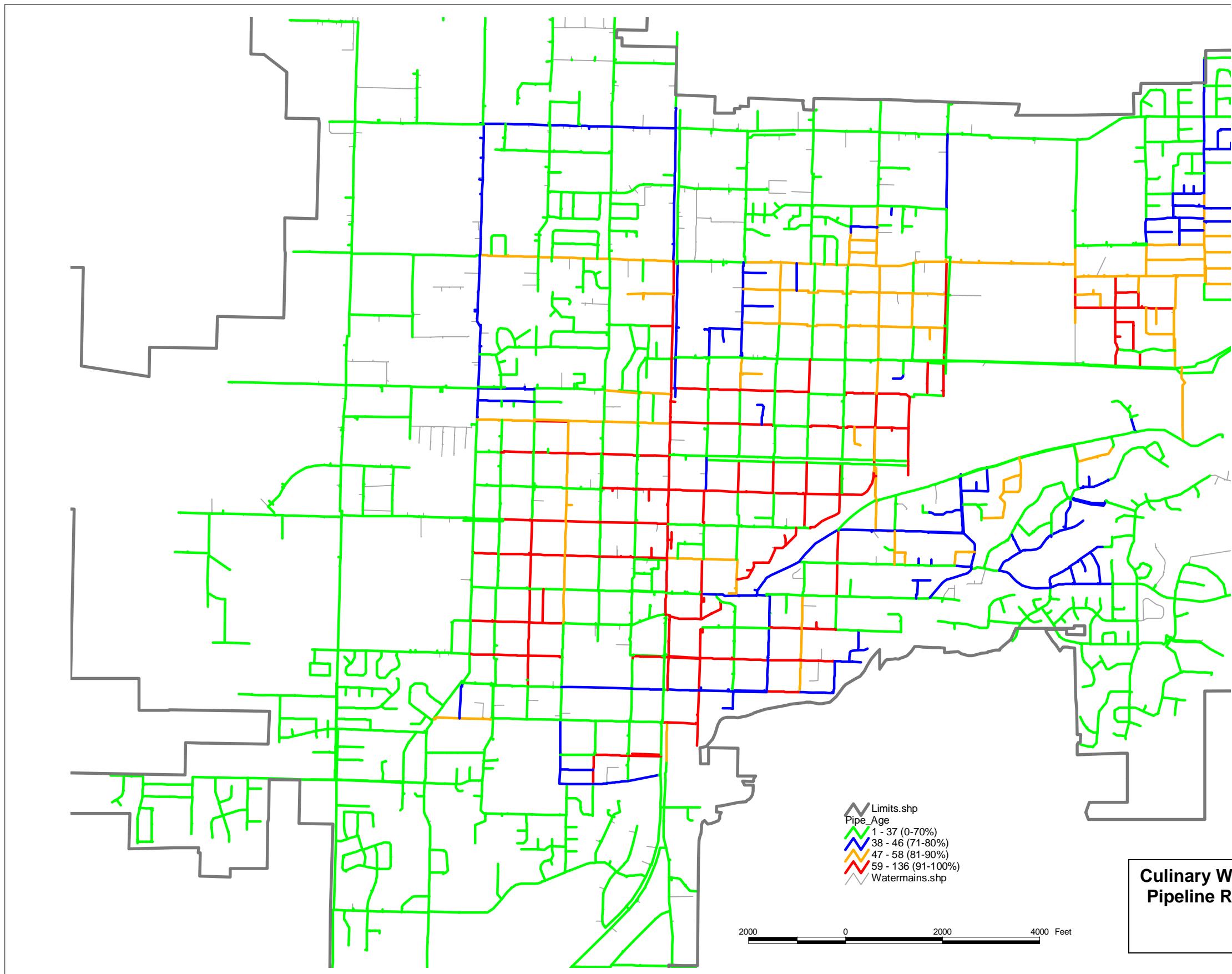
Figure 6.3 shows pipes by static pressure. Only 15% of the network has pressures less than 100 psi. Some 31% has pressures of 100 to 150 psi; and 47% of the network has pressures above 150 psi.

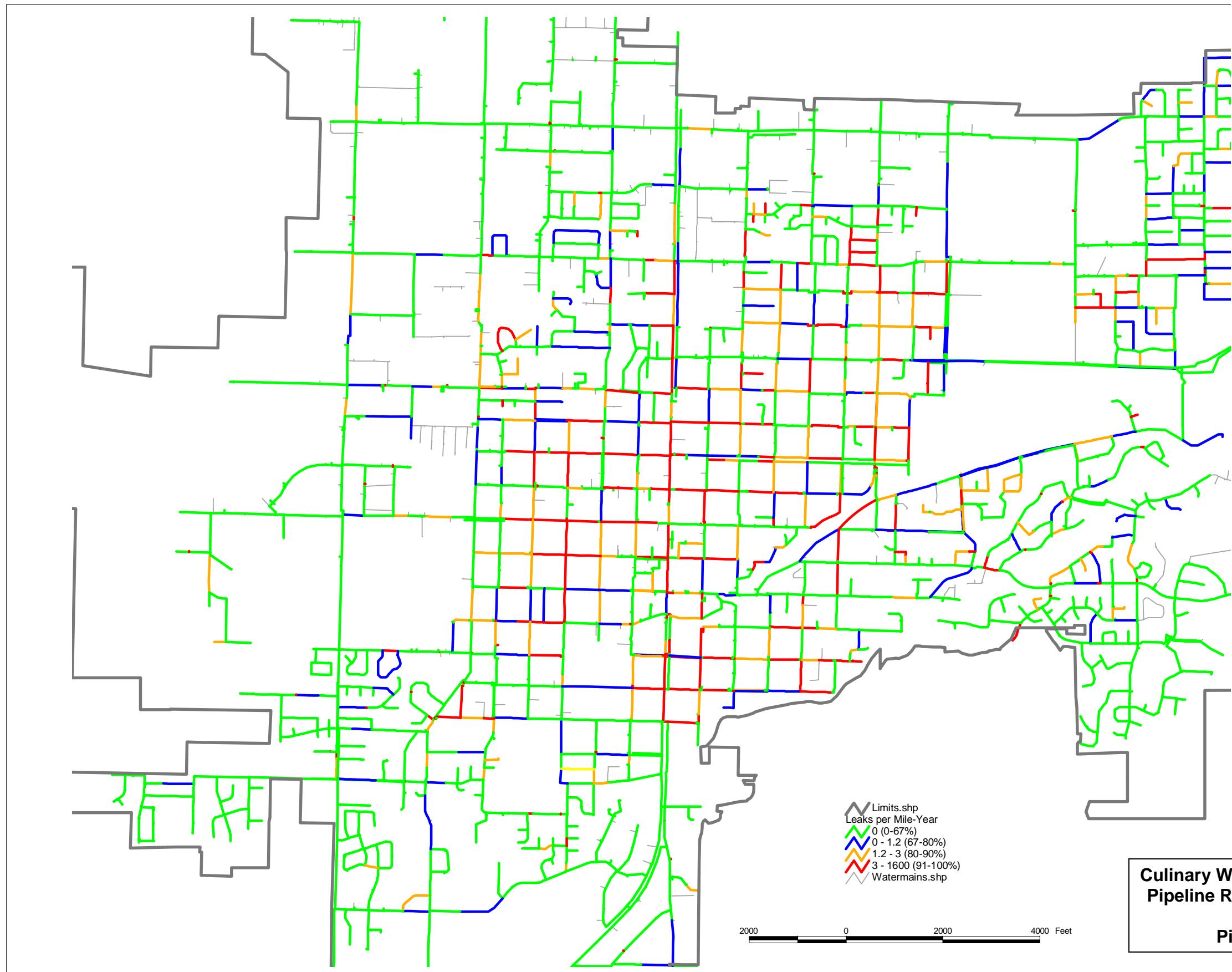
Figure 6.4 then combines the prioritization scores for pipe age, leaks/mile, and pressure with soil corrosion potential and weighting of scores at 1.0 for all three prioritization categories. The highest priority mains had combined scores of 8 to 10. These represent about 10% of the network and are shown in red. About 75% of the network was in the category for lowest priority, with combined scores of 1 to 5.

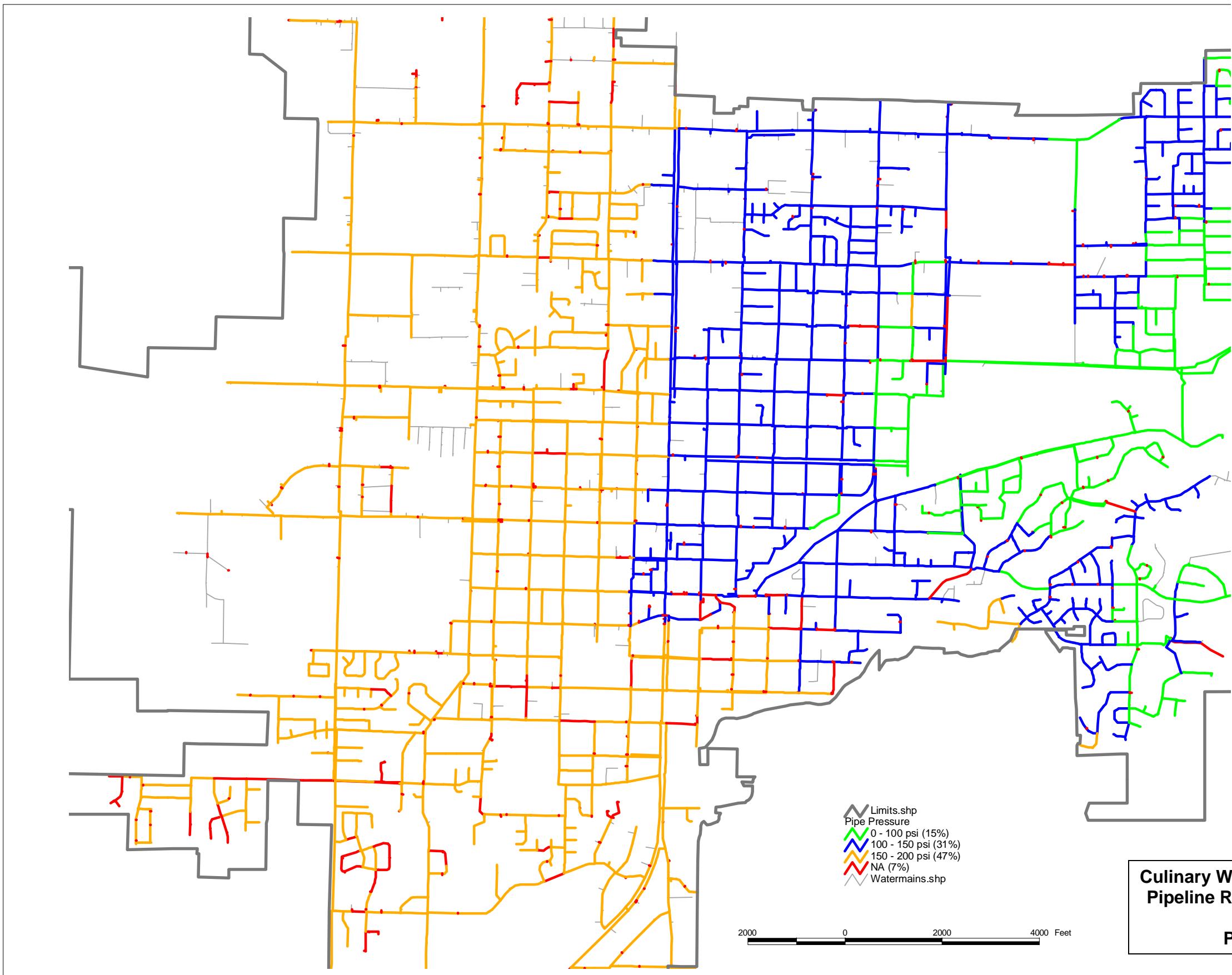
This figure indicates that highest replacement priorities are located primarily in the central part of the city. These are often in convenient packages of several blocks.

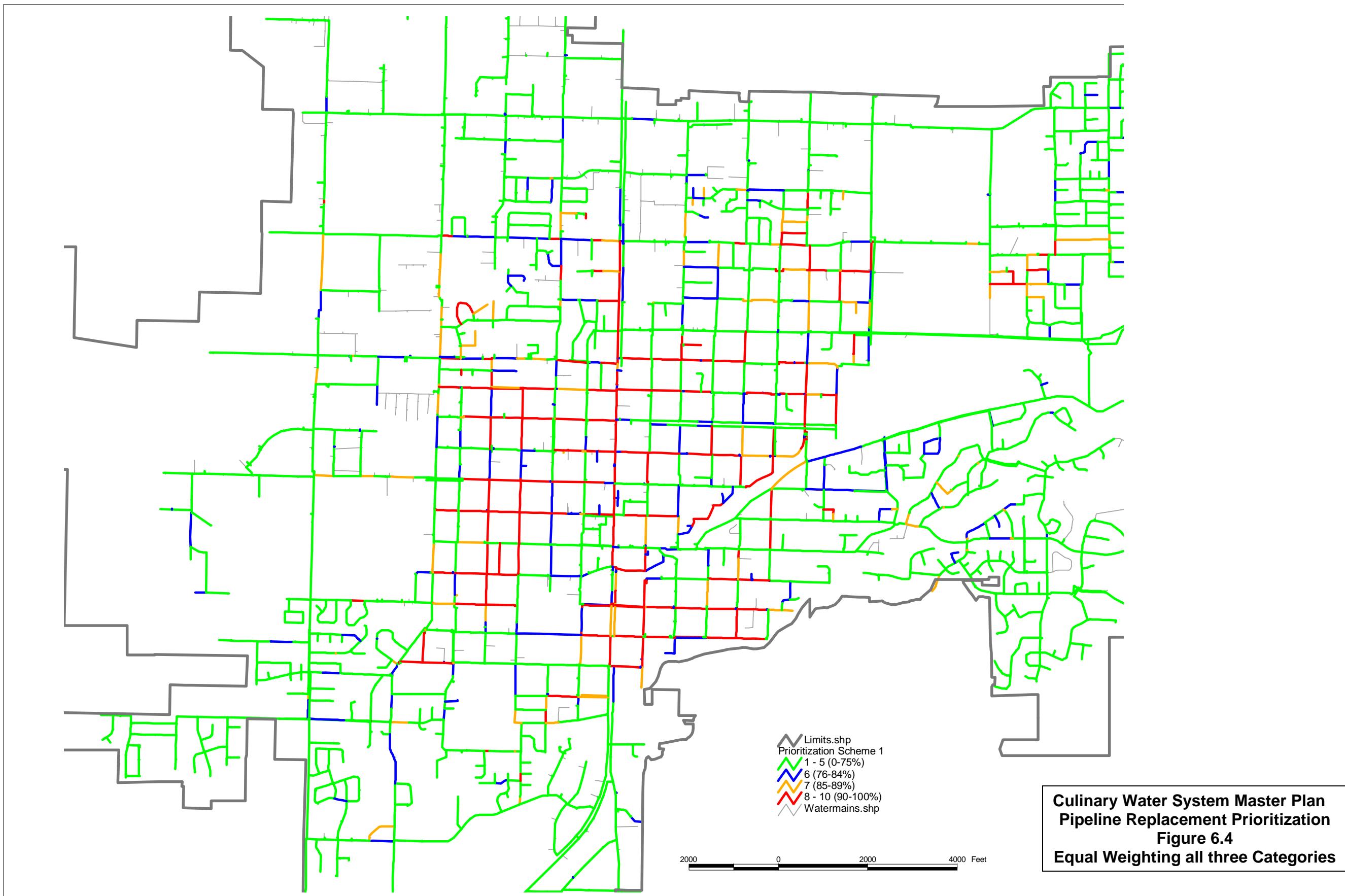
Incorporating Weighting to Pipe Physical Parameters

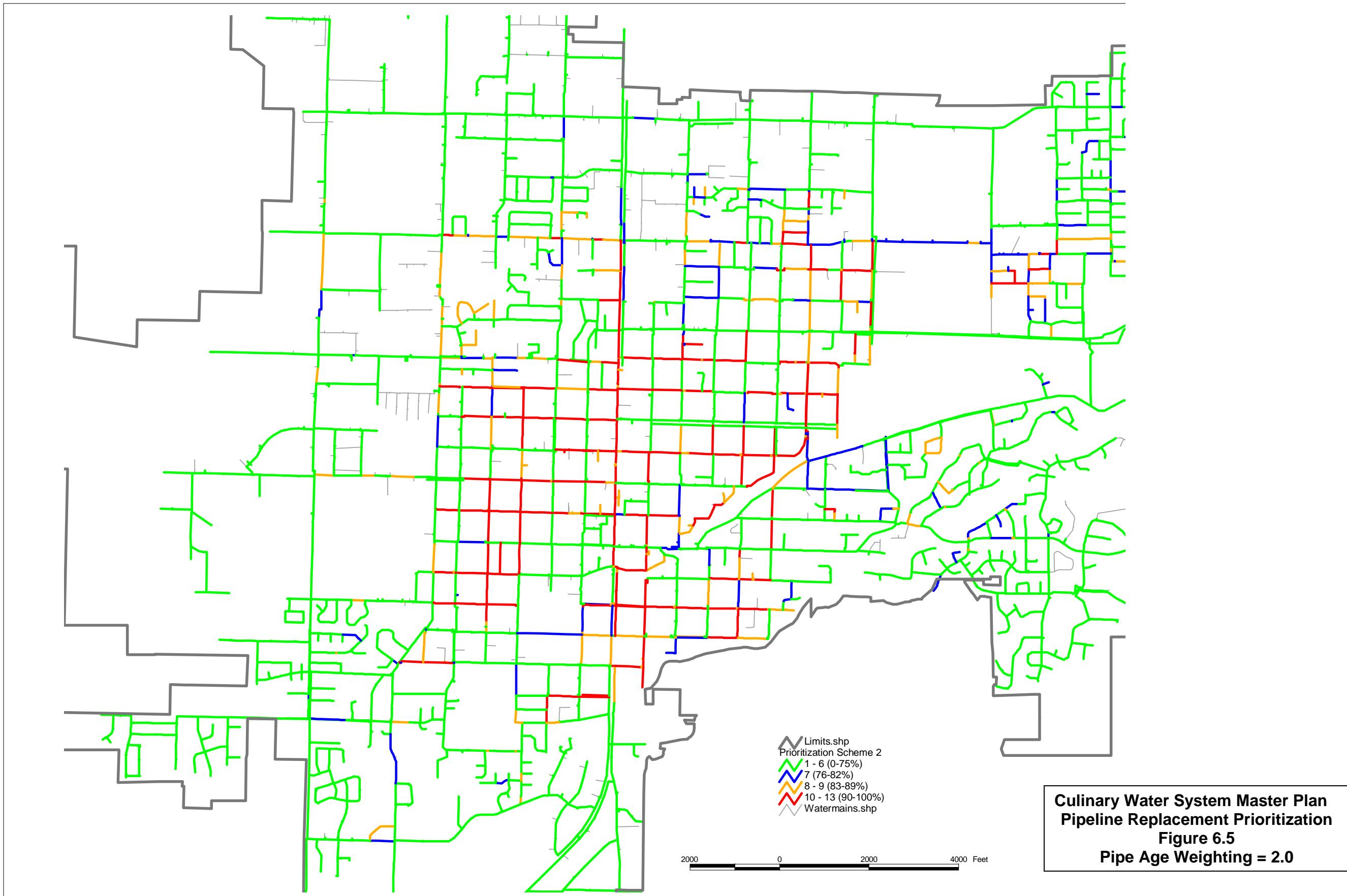
In Figure 6.5, pipe age scores have been given a weighting of 2.0. As expected, pipes of equal priority tend to occur along longer pipe segments.













Looking at the top 10% of priority pipes, this produces large packages of pipe replacement projects in the central part of the City.

In the next step in prioritization analysis, the pipe age is given a weighting multiplier of 2.0, and pipe leaks/mile is given a weighting multiplier of 3.0. The results of this calculation are shown in Figure 6.6. Pipes in the highest 10% of priority are shown in red. Similar to Figure 6.4, this weighting tends to cause shorter segments of high priority pipes interspersed among lower priority pipes.

These analyses have only looked at physical parameters, in order to identify patterns. Soil corrosion potential was not used because scores mostly uniform across the network, and would therefore not impact prioritization.

Incorporating Pipe Prioritization Categories for Pipe Criticality, Coordination with Other Projects, and Supervisor Input

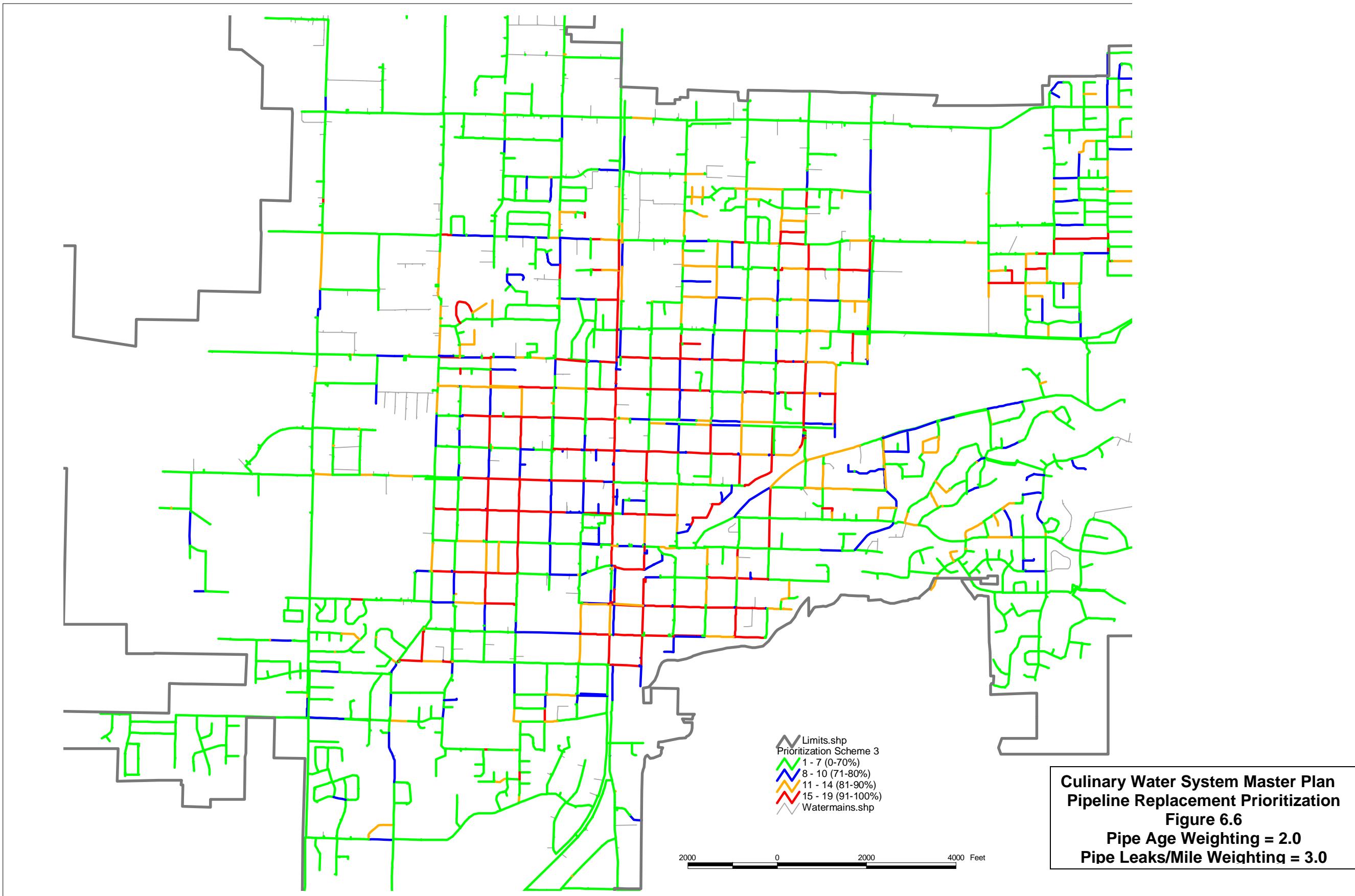
Next, pipe criticality, coordination with other projects, and supervisor input were incorporated into the analyses. These categories play a role in bringing some pipes forward in prioritization based on several criteria.

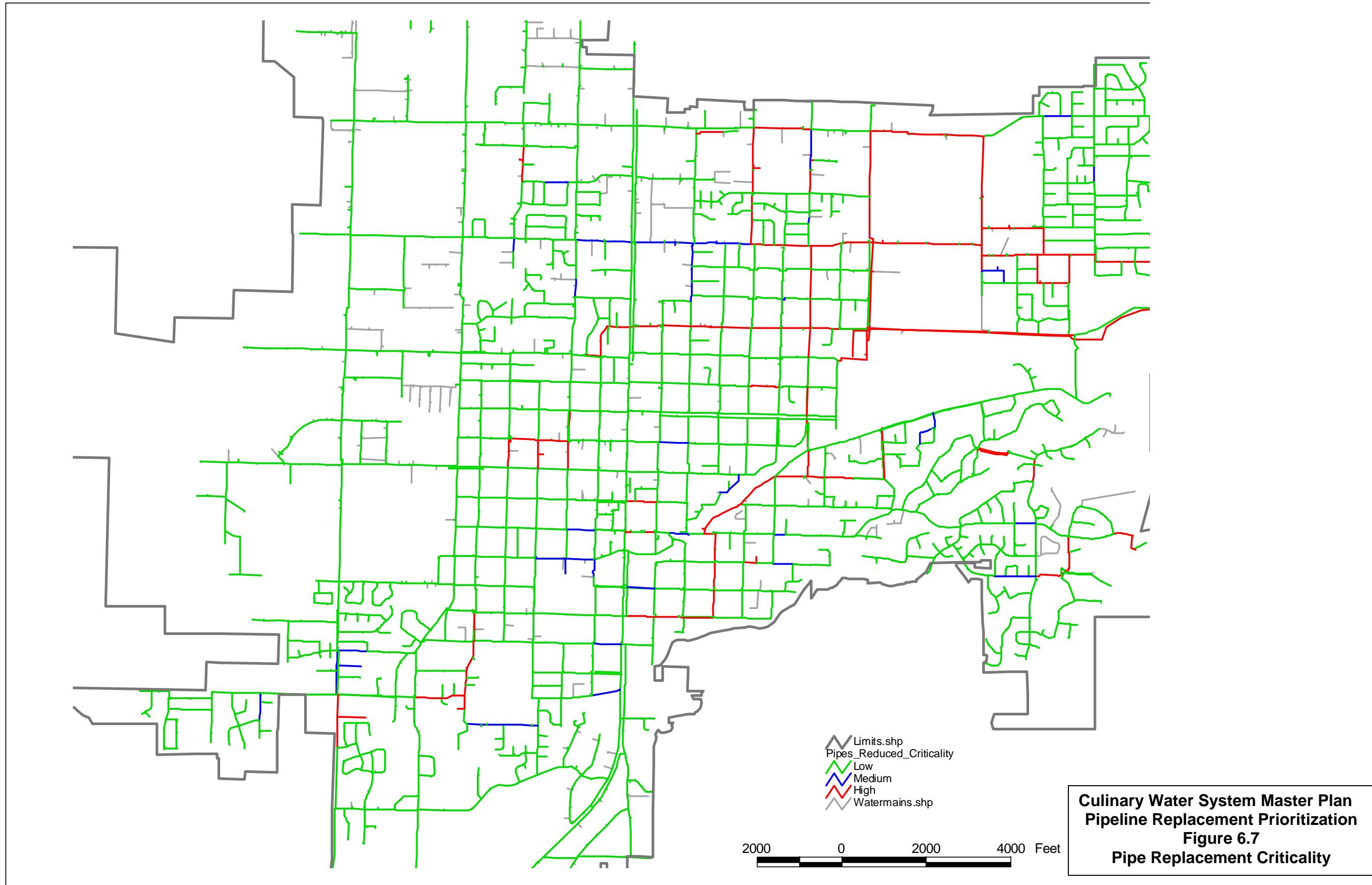
Figure 6.7 shows pipe criticality, which includes mains connecting critical water supply facilities and critical customer services as previously described. Figure 6.8 shows mains whose replacement can be prioritized based on coordination with other City projects. Figure 6.9 shows mains prioritized according to supervisor input.

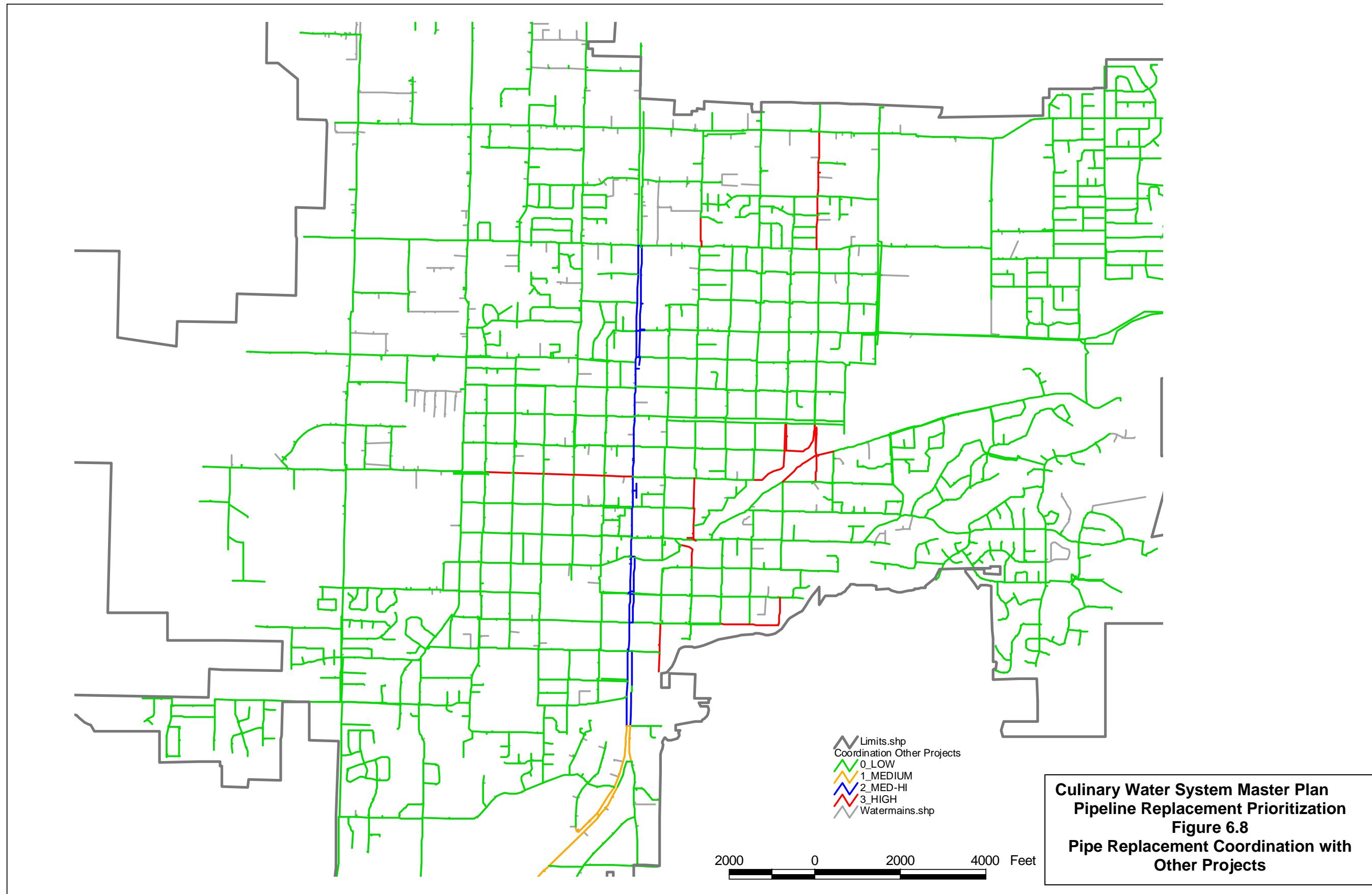
It should be pointed out that many of the mains given high priority in these categories are already scheduled for replacement within the next one or two years.

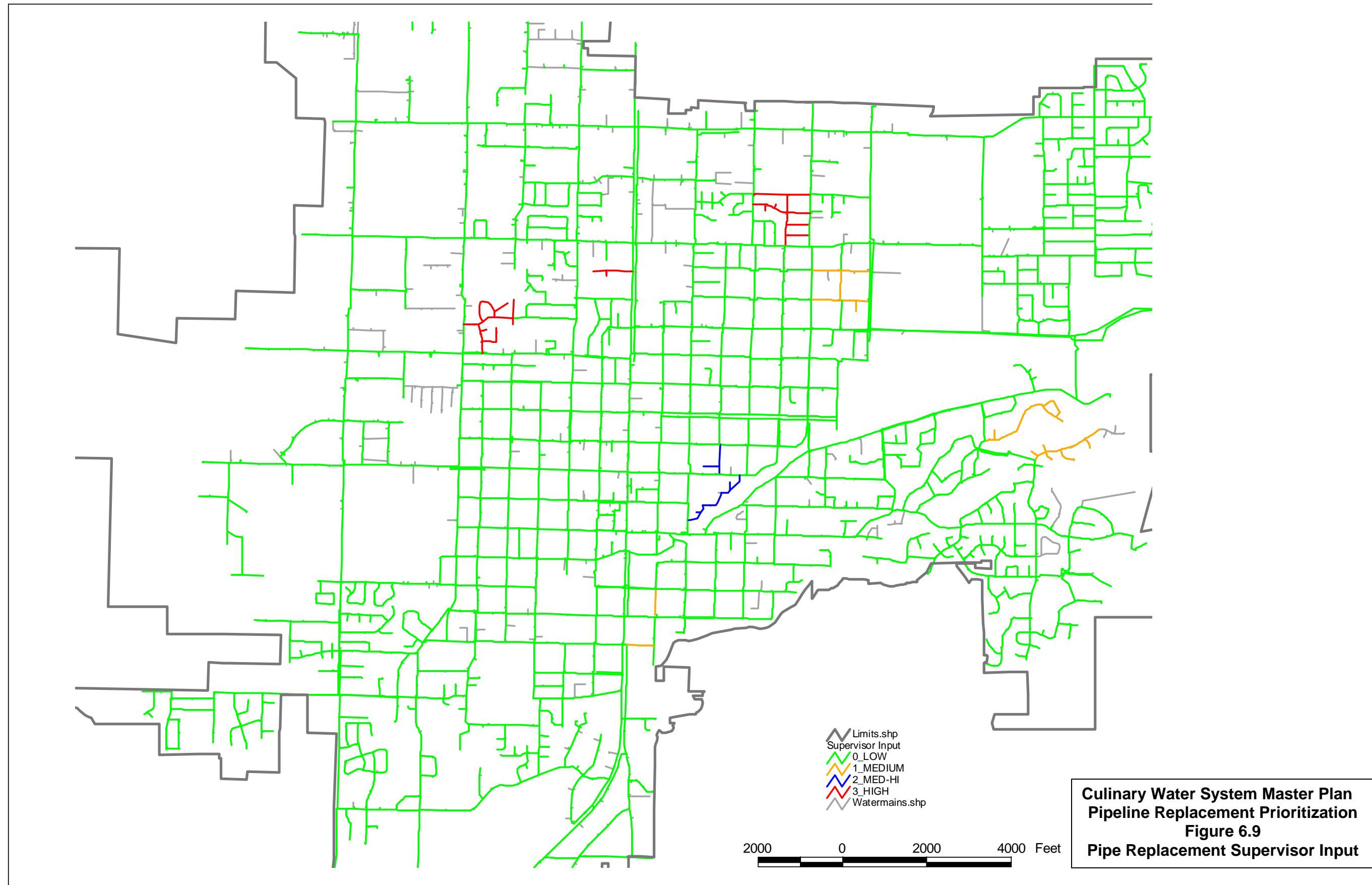
When criticality, coordination with other projects, and supervisor input are incorporated into the prioritization model illustrated in Figure 6.6, then it is possible to create a final ranking of weighted scores and quantify annual pipe replacement requirements based upon ranges of scores.

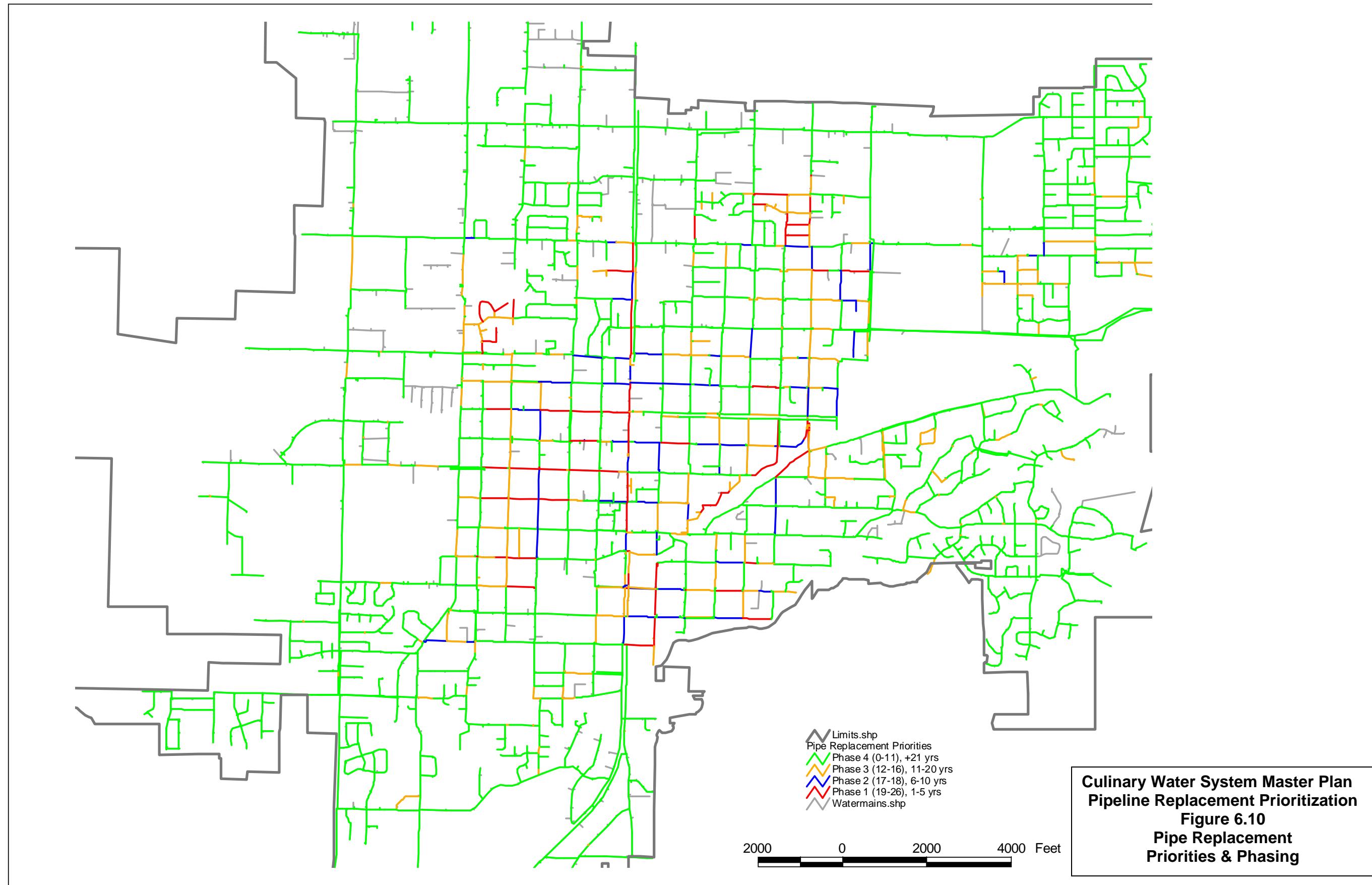
Figure 6.10 shows the proposed pipe replacement priorities and phasing. Phase I, over the first five years, covers all pipes with scores of 19 to 26; and Phase II, the next five years, covers pipes with scores of 17 to 18. After this phasing was developed for the next ten years, and then beyond.













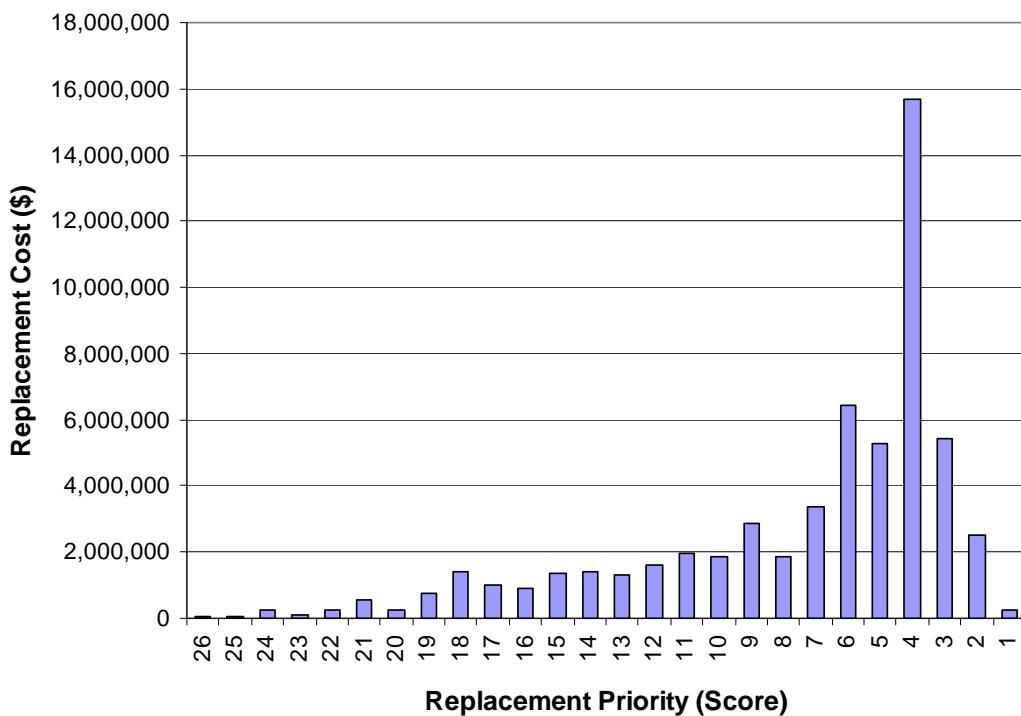
Phases I and II include convenient packages of pipe segments, primarily in the older part of the city, and a clear picture of pipe replacement priorities for the first ten years.

While these priorities are developed by the procedures discussed in this section, the actual grouping should not skip one small segment of lower-priority pipe in the middle of two larger higher-priority segments. This would create additional unnecessary disruption to City residents and businesses as future replacement needs come due.

Pipe Replacement Costs by Priority

As pointed out earlier, the intent of pipe replacement planning is to develop annual pipe replacement budgets based on a logical prioritization of pipe replacement needs. Figure 6.11 confirms that the prioritization scoring results in a cost effective basis for developing the pipe replacement plan, where highest priority pipes are replaced first.

Figure 6.11
Logan Pipe Replacement Costs by Priority



Given the prioritization of pipe replacement, the City of Logan can prepare phased replacement packages. Such packages will be adjusted over time according to the needs of the City using pipe replacement planning tools and available budgets.



6.4 Water Main Survival Curves

The AWWA publication “Quantifying Future Rehabilitation and Replacement Needs of Water Mains” gives life expectancy estimates from four US water utilities:

- Boston Water and Sewer Commission (BWSC)
- Fort Worth Water Department (FW)
- Los Angeles Department of Water and Power (LADWP)
- Philadelphia Water Department (PWD)

Water main survival curves were developed from each set of data. To these were added curves from an evaluation of water main survival in Bellevue, Washington. The importance of water main survival curves is their ability, using anticipated lifetimes based on historic data for each pipe type and comparisons with other utilities, to predict annual replacement rates necessary to maintain or improve the level of service.

Figures 6.12 and 6.13 present the survival curves for cast iron and ductile iron pipe. Note that Fort Worth assumes that at 50 years of age 100% of cast iron pipe is still in service, and 100% of ductile iron pipe is still in service at 60 years of age. At the other end of the scale, Bellevue High Deterioration assumes that at 50 years of age already some 23% of pipe will have to be replaced.

City of Logan data indicates that a significant portion of the network is 100 years or older, but that ductile iron pipe installed on the west side of town is failing after only 10-20 years. The Bellevue High Deterioration survival curve most closely approximates experience with the City of Logan network – in this curve, some pipe may last as long as 200 years, while there is also early failure of pipe due to a variety of factors.

Therefore the Bellevue High Deterioration survival curve best accounts City of Logan experience and will be used in pipeline replacement planning. The impact of using this survival curve is discussed later in this section.



Figure 6.12
Water Main Survival Curves for Cast Iron Pipe

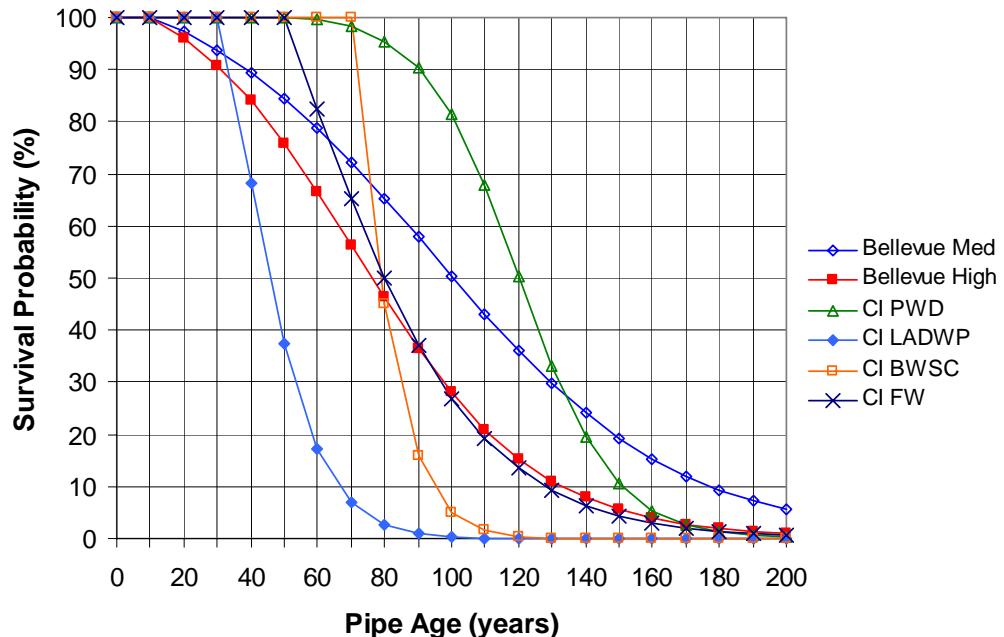
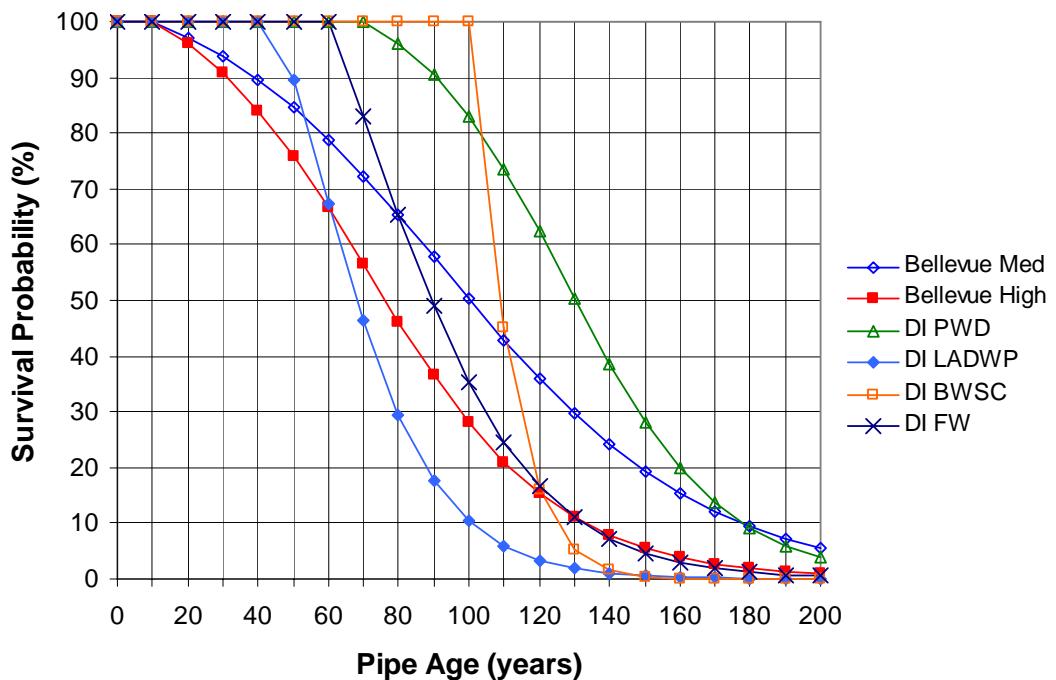


Figure 6.13
Water Main Survival Curves for Ductile Iron Pipe





6.5 Pipeline Replacement Unit Costs

Table 6.1 presents the unit costs which were applied when developing pipe replacement budgets. These unit costs were reviewed with City staff.

Table 6.1
Pipeline Replacement Unit Costs

Diameter (in)	Unit Costs in \$/ft				
	Construction Cost ¹	General Requirements ²	Contingencies ³	Professional Services ⁴	Total Unit Cost
1	16.00	1.60	5.30	3.40	26.30
1.5	21.00	2.10	6.90	4.50	34.50
6	48.00	4.80	15.80	10.30	78.90
8	54.00	5.40	17.80	11.60	88.80
10	62.00	6.20	20.50	13.30	102.00
12	70.00	7.00	23.10	15.00	115.10
14	77.00	7.70	25.40	16.50	126.60
16	86.00	8.60	28.40	18.50	141.50
18	95.00	9.50	31.40	20.40	156.30
20	101.00	10.10	33.30	21.70	166.10
24	121.00	12.10	39.90	26.00	199.00

¹1st Quarter 2006 construction costs from ENR Construction Cost Index.

²Mobilization, insurance (10%).

³Contingencies (30%)

⁴Engineering, legal (15%).

The total cost of replacing all network pipes – some 156.2 miles – would be about \$76.4 million (with 30% contingency included). While all pipes will eventually need replacing, based on the survival curves a fraction of pipe may last until 200 years of service.

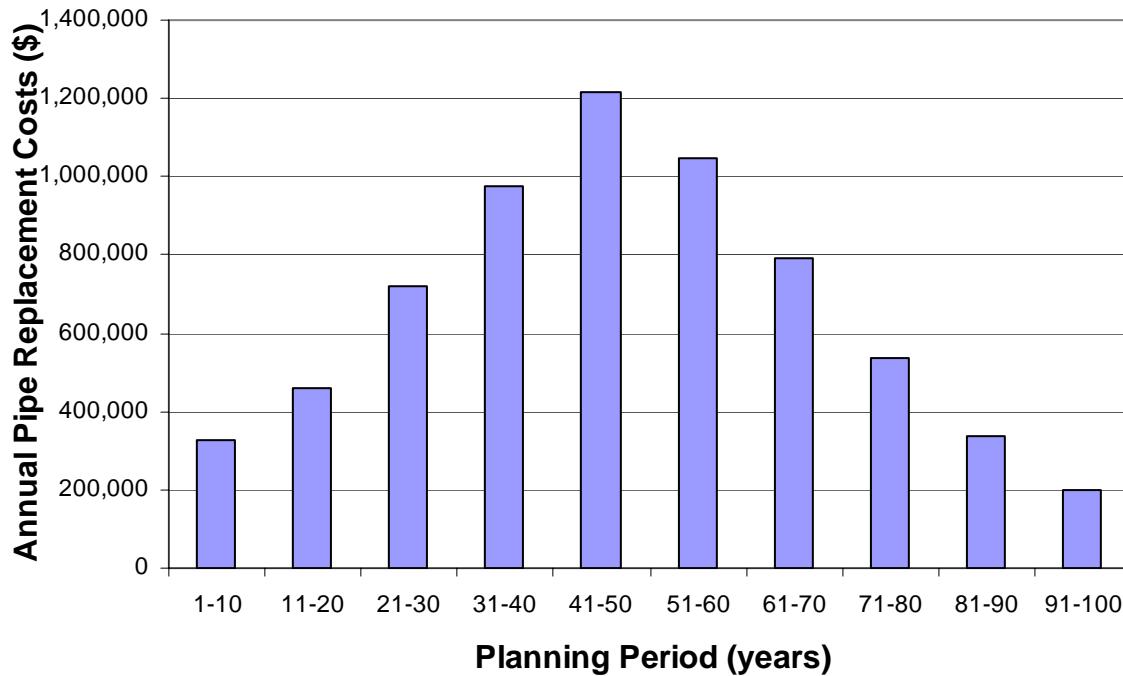
6.6 Pipeline Replacement Alternatives

Using the unit costs from section 6.5 above, and pipe age data, Black & Veatch prepared several pipe replacement alternatives based on a selection of water main survival curves. The curves employed and the consequences of their use are explained below.

LADWP Ductile Iron Pipe Survival Curve: In this pessimistic scenario, replacement costs for a 100-year study period would be \$66.1 million. Pipe replacement costs are lower at first but peak at \$1.22 million/year after 40 years, as shown in Figure 6.14.



Figure 6.14
Annual Pipe Replacement Costs – Pessimistic Scenario



Bellevue Medium Deterioration Rate Survival Curve: This is an optimistic scenario, where pipe replacement costs total \$44.8 million over 100 years. Pipe replacement costs are lower at the beginning and peak at \$0.52 million per year after 60 years. This is shown in Figure 6.15.

Fort Worth Cast Iron Survival Curve: This is a deferred cost scenario, where pipe replacement costs total \$61.1 million over 100 years, but are very low during the first 40 years and then peak at \$0.98 million per year after 50 years. This is shown in Figure 6.16.

Bellevue High Deterioration Rate Survival Curve: This is a middle-to-average cost scenario, with pipe replacement costs totaling \$55.3 million over 100 years. Annual costs are more even through the planning period, with a peak of about \$0.67 million per year after 40 years. This is shown in Figure 6.17.

Each water main survival curve and resulting pipe replacement alternative was discussed with City staff in order to select the alternative which best represents actual network conditions in Logan.



Figure 6.15
Annual Pipe Replacement Costs – Optimistic Scenario

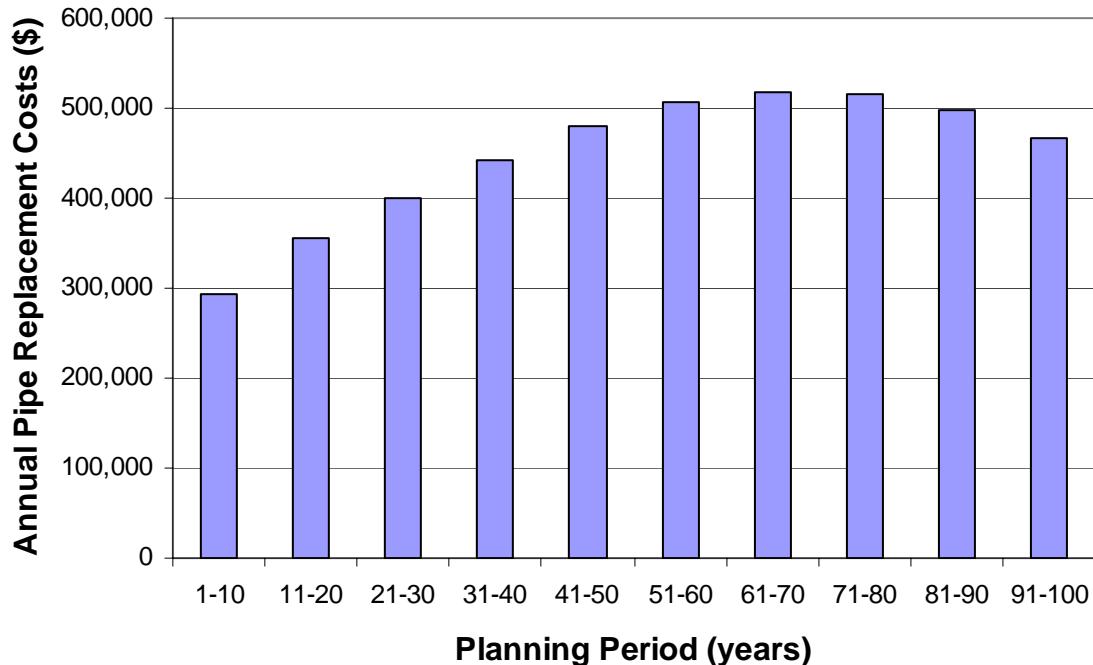


Figure 6.16
Annual Pipe Replacement Costs – Deferred Costs Scenario

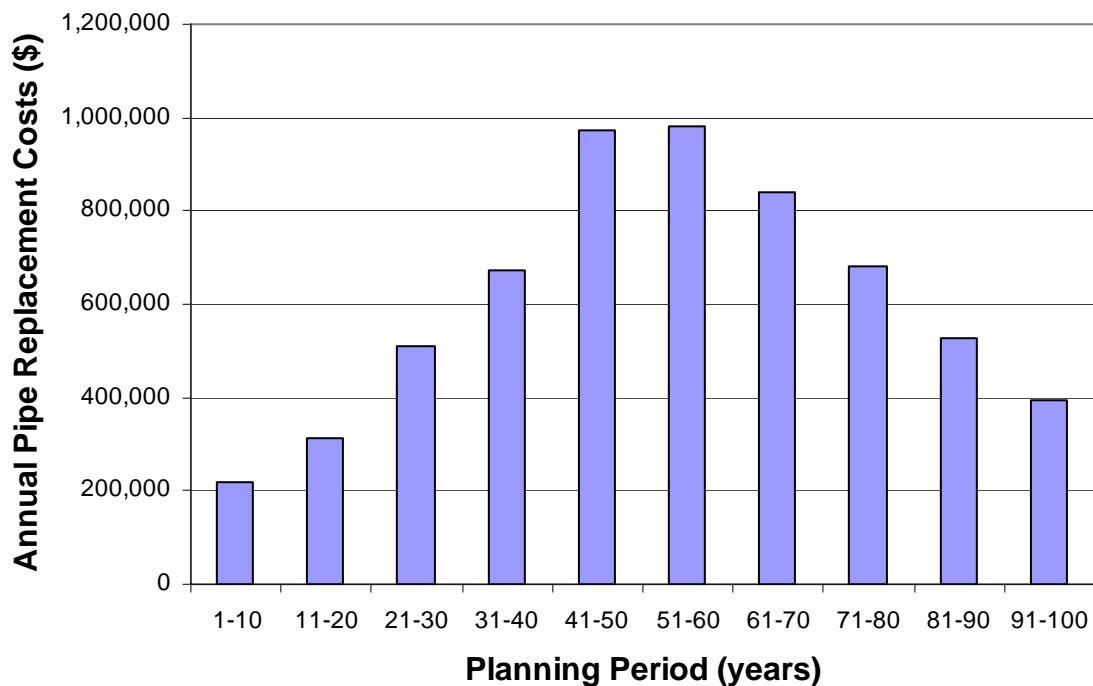
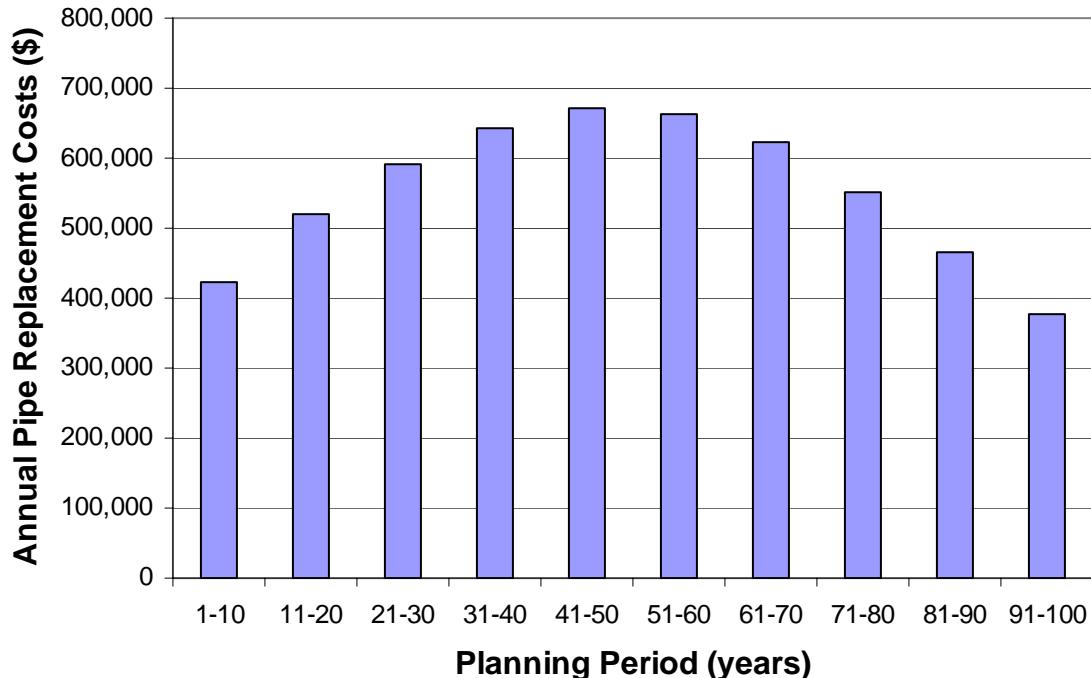




Figure 6.17
Annual Pipe Replacement Costs – Middle/Average Costs Scenario



It was determined that the curve from Bellevue High Deterioration best takes into account the effect of corrosive soils on cast iron and ductile iron pipe in Logan, as well as takes into account the middle/average and conservative pipe lifetimes seen in some areas of Logan, where pipe service life exceeds 100 years. Also, about 79% of total network pipe in Logan is cast iron or ductile iron.

This curve, used in Figure 6.17 above, also presents a phased pipe replacement that neither understates nor defers early pipe replacement needs and costs.

6.7 Pipeline Replacement Plan

City of Logan staff accepted the pipe replacement costs scenario modeled after the Bellevue High Deterioration survival curve. However, the City also noted that there was an \$8 million backlog of needed investments – not unusual in any large municipality – that needed to be added to the pipeline replacement plan. This backlog should be resolved during the first 20 years. The resulting plan is shown in Table 6.2 and Figure 6.18.

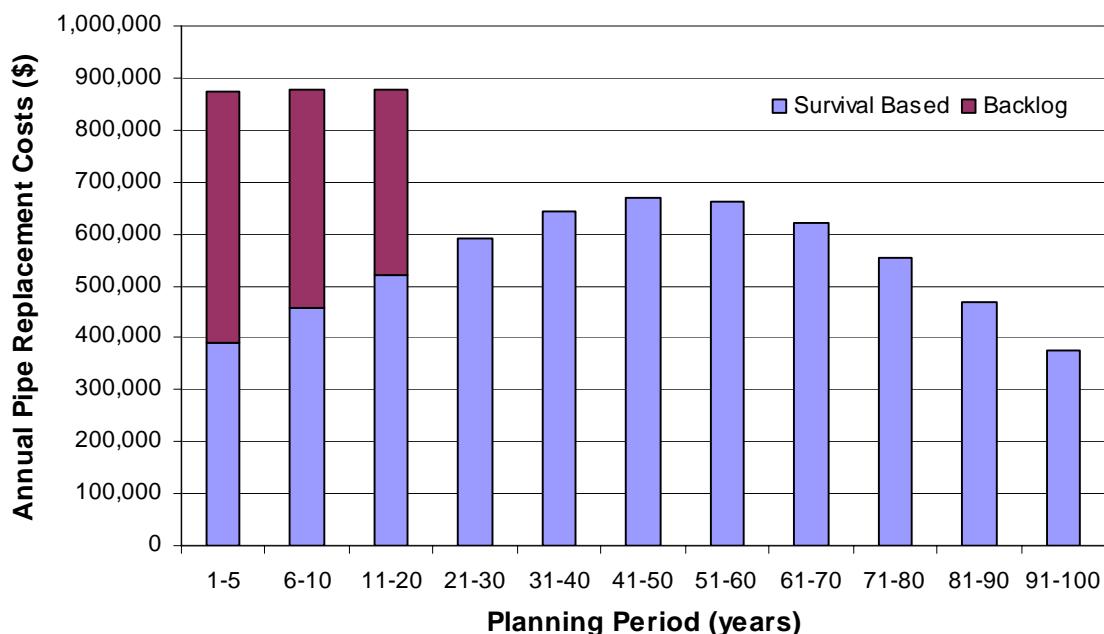


Table 6.2

Logan Pipeline Replacement Plan

Planning Period (years)	Planning Period Cost (\$ million)			Annual Cost (\$ million/year)		
	Backlog	Survival-based	Total	Backlog	Survival-based	Total
1-5	2.424	1.946	4.370	0.485	0.389	0.874
6-10	2.101	2.285	4.386	0.420	0.457	0.877
11-20	3.555	5.208	8.763	0.356	0.521	0.876
21-30		5.909	5.909		0.591	0.591
31-40		6.440	6.440		0.644	0.644
41-50		6.702	6.702		0.670	0.670
51-60		6.629	6.629		0.663	0.663
61-70		6.215	6.215		0.622	0.622
71-80		5.524	5.524		0.552	0.552
81-90		4.666	4.666		0.467	0.467
91-100		3.765	3.765		0.377	0.377
	8.080	55.287	63.368			

Figure 6.18
Logan Annual Pipe Replacement Costs





7.0 Regulatory Review

The purpose of the regulatory review is to determine if the water supply system is currently compliant with drinking water regulations, and determine if and how upcoming regulations may affect the system. The regulatory review therefore takes into account how the water supply system is currently operated, looks at historic water quality data, and addresses the relevance and impacts of current and pending Safe Drinking Water Act (SDWA) regulations.

The City of Logan has provided five years of annual water quality reports and some detailed water quality information in order to conduct this review. The Utah Division of Drinking Water (DDW) supplied detailed water quality information from their water quality database, spanning over about 30 years of data. The annual reports and DDW information are provided in the Appendix.

7.1 Description of System Operations

The water supply sources consist of the DeWitt Springs (DDW identification number 01) located in Logan Canyon, and four wells at several locations in the valley below:

- Crockett Avenue Well, ID no.02
- 200 East Center Well, ID no.03
- 700 North 600 East Well, ID no.04
- Willow Park Well, ID no.05

The City of Logan considers both the springs and the wells as groundwater not under the influence of surface water, and this has been confirmed by DDW.

Water from DeWitt Springs is disinfected using gaseous chlorine and then conveyed by pipeline five miles through the canyon before entering the 6 MG storage reservoir at the mouth of the canyon. From the 6 MG reservoir (called GC 1/2), water flows into the water distribution network. Water users in Logan Canyon that take water directly from this pipeline include cabins (beginning at 1½ miles downstream), the Zanavu Lodge (two miles downstream), and the US Forest Service (pumped directly from DeWitt Spring to Malibu Campground two miles upstream).



Water from the wells is pumped without any disinfectant directly into the network.

The City is primarily supplied by water from the DeWitt Springs. This is supplemented in the summer by water from the wells in order to meet peak demands and maintain sufficient system pressure.

Water quality monitoring is carried out for points within the network and at the sources as required by State and Federal regulations.

7.2 Groundwater Source Protection

The City of Logan has prepared a series of ground water source protection plans in 1996, a more detailed review of the DeWitt Springs source in 1997, and updates in 2002.

The DeWitt Springs source has adequate protection from contamination, and no sources of potential contamination were identified during preparation of the source protection plan. The DeWitt Springs source is also well protected from future sources of potential contamination within the area.

The four wells, listed in the DDW database as ID nos.02 to 05, are located in urban areas where there are potential sources of contamination. Each well is protected from contamination by a concrete surface seal extending 100 feet into the ground.

According to the Source Protection Plans, the main sources of potential contamination of the wells are residential areas within their protection zones. This is because residential use of pesticides, fertilizers and other chemicals is harder to regulate. Other sources of potential contamination include commercial establishments, underground storage tanks, the Logan Regional Hospital, the city cemetery, Utah State University, park and fair grounds, and the local zoo.

The 2002 plan update determined that these wells are adequately protected from present and future sources of potential contamination, and therefore that the susceptibility of the drinking water wells to contamination is low.



7.3 Current and Pending SDWA Regulations

The USEPA has many different regulations in various stages of preparation, promulgation, and revision. This dynamic process takes years from the time a regulation is proposed until it becomes effective, often after significant revisions.

The regulatory review is based on current or pending regulations. It is not possible to predict precisely what additional regulations will be imposed beyond those currently under preparation, in particular because those currently under development are still subject to revision prior to being adopted.

Table 7-1 provides an overview of the status of current and pending regulations. **Those regulations marked in bold are most applicable to the five Logan source waters, and will be discussed in the following subsections.**

Table 7-1
Schedule for Promulgation of Pertinent SDWA Regulations
(Current as of April 2007)

Regulation	Proposed	Final	Effective
Trihalomethanes	Feb 1978	Nov 1979	Nov 1983
Fluoride	Nov 1985	Apr 1986	Oct 1987
Phase I VOCs	Nov 1985	Jul 1987	Jan 1989
Surface Water Treatment Rule	Nov 1987	Jun 1989	Jun 1993
Coliform Rule revisions ¹	Nov 1987 2006	Jun 1989 Aug 2008	Dec 1990
Lead and Copper Rule minor revisions minor revisions	Aug 1988 Apr 1998 Jul 2006	Jun 1991 Jan 2000 Jul 2006	Jan 1992 Jan 2001 Jul 2006
26 SOCs² and 7 Phase II Inorganics	May 1989	Jan 1991	Jul 1992
Phase V SOCs and Inorganics	Jul 1990	Jul 1992	Jan 1994
Barium, Pentachlorophenol Phase II MCLs	Jan 1991	Jul 1991	Jan 1993
Information Collection Rule	Feb 1994	May 1996	completed
Sulfate	Dec 1994	not regulated	Jul 2003
Interim Enhanced Surface Water Treatment Rule Stage 1 – Long Term ESWTR Stage 2 – Long Term ESWTR	Jul 1994 Apr 2000 Aug 2003	Dec 1998 Jan 2002 Feb 2006	Jan 2002 ³ Jan 2005 ⁴ Feb 2006 ⁵
Stage 1 Disinfection/Disinfection Byproducts Stage 2 Disinfection/Disinfection Byproducts	Jul 1994 Aug 2003	Dec 1998 Feb 2006	Jan 2002 ^{3,4} Feb 2006 ^{5,6}
Source Water Protection Program – Guidance	Aug 1997	completed	completed
Consumer Confidence Reports Rule	Feb 1998	Aug 1998	Sep 1998
Operator Certification – State Guidance	Mar 1998	Feb 1999	Feb 2001



Table 7-1
Schedule for Promulgation of Pertinent SDWA Regulations
(Current as of April 2007)

Regulation	Proposed	Final	Effective
Unregulated Contaminant Monitoring 1 ⁷	Feb 1999	Sep 1999	Jan 2001
Unregulated Contaminant Monitoring 2	Aug 2005	Jan 2007	Jan 2008
Radon⁸	Nov 1999	2009	2010
Radionuclides, Phase III (except Radon)	Apr 2000	Dec 2000	Dec 2007
Filter Backwash Recycle Rule	Apr 2000	Jun 2001	Dec 2003
Ground Water Rule	May 2000	Oct 2006	Dec 2009
Arsenic Rule	Jun 2000	Jan 2001	Jan 2006
Chlorine Gas as Restricted Use	Sep 2000	final notice delayed	
Contaminant Candidate List 1	Jul 2003	Jul 2003	completed
Contaminant Candidate List 2	Apr 2004	May 2005	May 2008
Aldicarb, Aldicarb Sulfoxide, Aldicarb Sulfone	Aug 2004	Dec 2005	Dec 2008 ⁹
Effluent Guidelines for Water Treatment Plants	2006	2007	2010

¹Revised Total Coliform Rule may become Distribution System Rule.

²MCL (Maximum Contaminant Level) and MCLG (Maximum Contaminant Level Goal) for Atrazine to be reconsidered.

³For Public Water Systems serving > 10,000 persons.

⁴For Public Water Systems serving < 10,000 persons.

⁵Monitoring begins.

⁶Running annual average (LRAA) to be completed at each sampling location incl. sites with high DBPs.

⁷Tiered monitoring approach, pending availability of analytical methods.

⁸Delayed, target date now set.

⁹Assumes regulation in effect three years after final promulgation.

7.4 Rule-by-Rule Compliance Review

Trihalomethanes

The four trihalomethanes (THMs) are known carcinogens and suspected of causing other health difficulties if present in water at high concentrations. The original level set for these contaminants was a total of 0.10 mg/L. This MCL has since been revised by the Stage 1 Disinfectants/Disinfection Byproducts Rule (DBPR) to 80 µg/L (0.080 mg/L), and will be further revised under the Stage 2 DBPR.

The THMs in the City's distribution system are seldom greater than 1 µg/L (0.001 mg/L), which is an extremely low level. These contaminants will again be mentioned when the Stage 1 DBPR and the Stage 2 DBPR are discussed later in this section, but obviously, Logan will not have any difficulty meeting THM requirements.



Fluoride

Fluoride has a maximum contaminant level, MCL, of 4 mg/L for public health protection against skeletal fluorosis. However, fluoride at lower levels, 1 to 1.5 mg/L, provides dental benefits by preventing tooth decay, especially in children. Thus, EPA has set a secondary MCL of 2 mg/L and systems that exceed this level must provide public notice to their customers.

The water sources for Logan have low levels of fluoride, and Cache County voters chose not to require fluoride addition as a preventive measure for tooth decay. The levels of fluoride in the City's distribution system are on the order of 0.15 to 0.2 mg/L which is well below the MCL.

Phase I VOCs

Most organic chemicals are volatile to some degree, and USEPA selected eight volatile organic chemical (VOCs) to be regulated under the Phase I Rule: benzene, vinyl chloride, carbon tetrachloride, trichloroethylene, p-dichlorobenzene, 1,1-dichloroethylene, 1,1,1-trichloroethane, and 1,2-dichloroethane.

All of these compounds have been recorded as being less than detectable levels in this water, and a violation of this Rule could only occur if the source waters became contaminated in the future.

Coliform Rule

The Coliform Rule allows for up to 5 percent of the monthly samples collected from within the City's distribution system to be positive for coliforms. Samples are run on a presence/absence basis and are not enumerated for routine testing.

Four samples tested positive for total coliforms presence in the year 2001 (representing a violation), two in 2002, one in 2003, and none thereafter. In each case the City conducted follow-up testing (three samples) without detecting coliforms. While it is possible the samples were contaminated (non-professional staff were used by Bear River Health Department for sampling), the City should be vigilant and continue to monitor this situation.



Lead and Copper Rule

The original LCR was promulgated in 1991, and established action levels if 10 percent of the lead concentrations exceeded 0.015 mg/L, or if 10 percent of copper concentrations exceeded 1.3 mg/L. Samples under this regulation were to be collected at the homeowner's tap with the homes to be identified under a series of requirements as dictated in the Rule. If action levels are exceeded, treatment actions such as corrosion control, lead line replacement, etc. must be taken.

Both lead and copper are consistently below the action levels mentioned above, with the 90th percentile lead level being in the 0.005 mg/L range and the 90th percentile copper level being about 0.20 to 0.30 mg/L. The City's water easily meets the requirements of this Rule.

Phase II and Phase V Organics

The Phase II and Phase V SDWA regulations, combined, list MCLs or treatment techniques for 44 synthetic organic chemicals (SOCs).

Testing to date indicates that the concentrations of these contaminants are consistently below detectable levels; exceeding their MCLs is not anticipated.

Phase II and Phase V Inorganics

Phase II and Phase V SDWA regulations, combined, list MCLs for 12 inorganics, although the MCL for nickel, was later remanded. Nitrate is the primary inorganic chemical of concern and it is measured for annually within the distribution system.

Nitrate levels in the water distribution network are generally 1.0 mg/L as nitrogen or less, considerably below the MCL for nitrate which is 10 mg/L as nitrogen. This data indicates that nitrate contamination is not a problem.

Stage 1 Disinfectants/Disinfection Byproduct Rule

The Stage 1 DBPR lowered the MCLs for trihalomethanes (THMs) to 80 µg/L, and established an MCL for five haloacetic acids (HAA5) at 60 µg/L. Additionally, this Rule required a certain percentage of total organic carbon (TOC) removal if the water system did not meet one of six exemptions from the TOC removal requirement. One exemption requirement is that THMs and HAA5 must be less than 40 µg/L and 30 µg/L, respectively, when using only chlorine. This is known as the 40/30 exemption.



The Logan system appears to meet this requirement: DDW confirms that Logan meets the requirements and has placed Logan on the 40/30 exemption list.

In addition to the new MCLs, the Rule also established a limit for disinfectants. Chlorine, which is dosed at DeWitt Spring, must not exceed a concentration of 4.0 mg/L.

Chlorine demand is reportedly very low. Therefore the City doses only about 0.4 mg/L of gaseous chlorine at DeWitt Spring, to keep chlorine residuals of 0.17-0.22 mg/L in the Golf Course reservoirs, and a disinfectant residual in the network.

Stage 2 Disinfectants/Disinfection Byproduct Rule

Like the Stage 1 DBPR, the Stage 2 D/DBPR applies to all community water systems that do not add a disinfectant other than UV. The MCLs for THMs and HAA5 are maintained at 80 µg/L and 60 µg/L, except that new locations that reflect the highest levels of these DBPs shall be established for most systems. This will be based on an Initial Distribution System Evaluation (IDSE) to identify alternate monitoring locations.

DBP levels in the Logan distribution system are so low (THMs from non-detect to 6.7 µg/L), that Logan has been exempted by Utah DDW from conducting the IDSE. Therefore the City can continue the current level of distribution system monitoring without having to meet stricter requirements.

Consumer Confidence Reports Rule

Each public water system is required to report certain water quality information to its customers annually. The City is in compliance with this Rule. However, the City's website should clearly identify where a copy of this report can be obtained.

Radon

The Radon Rule has been delayed, and the MCL for this contaminant has not yet been decided. The draft rule proposed an MCL of 300 pCi/L, and an alternate MCL of 4,000 pCi/L. Use of the 4,000 pCi/L MCL was to be contingent upon the adoption of a multimedia mitigation (MMM) program by either the state or the City. The status of this MCL is unknown. There was considerable debate over this limit as well as an evident reluctance by both states and communities to develop an MMM program.



If the MCL is set in the conventional manner at 300 pCi/L, some of the water sources currently used by the City may be in violation. Substantial radon data was not located, but one table issued by Tsai, Pierce, and Newman of the Utah Department of Health lists the radon level at Logan to be 388.3 pCi/L. It was not possible to identify which of the five City sources was sampled, but higher levels were detected for cities south of Logan. This data was collected in June to October of 1988, so current levels are unknown.

If the above data is representative and accurate, the City's water sources may be in violation if the lower MCL target for radon is established.

It is recommended that sampling for radon at each water source be done quarterly for one year. This data will then prepare the City to take any necessary action when the final rule is published should the radon level exceed the new MCL.

Radionuclides (Radon excluded)

Radionuclides are regulated by alpha and beta screening tests. Alpha particles have an MCL of 15 pCi/L and beta particles have an MCL of 4 millirems per year. There is also a combined radium 226 and 228 MCL of 5 pCi/L, and a uranium MCL of 30 µg/L.

Alpha particles range from non-detectable to 10 pCi/L in these source waters; however, most readings are < 2 pCi/L. Beta particles have only rarely been detected. At low gross alpha levels (< 5 pCi/L), monitoring for radium and uranium is not required, so data for these contaminants is not available. Meeting the MCLs for these radionuclides does not appear to be an issue.

Ground Water Rule

The Ground Water Rule (GWR) was finalized in October 2006, and triggered source water monitoring will be effective December 1st, 2009. The GWR includes the following requirements:

- Periodic sanitary surveys conducted by the State to identify any system deficiencies.
- Source water microbial monitoring for systems that do not provide treatment conditions that will result in 4-log virus removal/inactivation, and that experience a positive coliform sample during routine distribution system monitoring that is not subsequently invalidated.
- Any significant deficiencies must be corrected.
- Systems that disinfect must achieve and demonstrate a 4-log inactivation or removal of viruses in order to avoid the source water monitoring requirements listed above.



All the source waters used by Logan will be subject to the Ground Water Rule. Logan employs chlorine disinfection at the DeWitt Spring in order to maintain a disinfectant residual throughout the network. No disinfectant is employed at the four wells, nor is there chlorine boosting elsewhere in the system.

Based on current data and information, the only element of this Rule that will clearly affect Logan is the need to document that water from the chlorinated source at DeWitt Springs achieves at least a 4-log inactivation of viruses before reaching the first user.

The City doses 0.40 mg/L free chlorine at DeWitt Spring, and there is still a residual of 0.20 mg/L five miles downstream at the Golf Course Reservoir (GC 1/2) at the mouth of Logan Canyon.

The first users are:

1. The US Forest Service, which installed their own pump at DeWitt Spring to convey water to the Malibu Campground some two miles upstream.
2. Cabins along the pipeline route, beginning at 1½ miles downstream.
3. Zanavoo Lodge, which is just less than two miles downstream.

Black & Veatch has prepared a calculation for viral inactivation in the pipeline for the location where the first cabins take water from the pipeline, which is at station 81+50. The calculation is based on the USEPA CT model.

The following conservative conditions are assumed based on information from the City: water temperature of 4.4°C and pH of 7.70. We have assumed a flowrate in the pipeline of 15 mgd. With a diameter of 36" from DeWitt to station 59+20, and 24" to the first cabins, the detention time in the pipe is 35 minutes.

Using the USEPA model, there would be 3.3 logs inactivation of viruses if the free chlorine concentration at the cabins is 0.2 mg/l; and there would be 4.9 logs inactivation if the free chlorine concentration at the cabins is 0.3 mg/l.



If we take into account a total detention time in the pipe of 70 minutes and assume a straight-line reduction in chlorine concentration, then we can interpolate a free chlorine concentration at the cabins of about 0.3 mg/l, which would comply with the 4-log inactivation requirements of the Ground Water Rule. The City should sample for free chlorine concentration at this location in order to verify this conclusion.

Regarding the Malibu Campground, the US Forest Service has its own pump and pipeline at DeWitt Spring. They pump to a storage tank, where they have their own chlorine booster equipment. The US Forest Service is therefore a water purveyor, and it is their responsibility to maintain the necessary chlorine residual to meet the requirements of the Ground Water Rule.

It is presumed that this new rule will not require the four City wells to be disinfected – unless the State sanitary survey deems them vulnerable. This needs to be verified after the final rule is promulgated. However, in conjunction with construction of the new DeWitt pipeline, chlorination at the wells will likely be required to maintain chlorine residuals in the water distribution network during construction.

Arsenic

The new arsenic regulation has lowered the MCL for this contaminant to 10 µg/L from the previous level of 50 µg/L.

DDW data shows arsenic concentrations for DeWitt Springs and three of the wells ranging from non-detect to a maximum of 2 µg/L, which is well below the regulation limit. However, no information was available on arsenic levels from the Crockett Well. The City should have an analysis performed to confirm that the Crockett Well also will comply with the Arsenic Rule.

Chlorine Gas as Restricted Use

The intent of this rule was to improve operations personnel safety. However, the rule would have required 24-hour operator presence at each chlorine dosing station. This would be a significant economic hardship for many small communities, and the rule in its original form has been tabled indefinitely.



Aldicarb, Aldicarb Sulfoxide, and Aldicarb Sulfone

Presently there is no limit for these contaminants. Based on the high quality water sources used by the City and the other organic data available from prior sampling, violating any future MCL for these contaminants would not be expected.

7.5 Additional Future Regulations

Some additional future regulations are expected. These undefined regulations would include rules for atrazine, a distribution system rule, limits for MTBE and perchlorate, and possibly rules for endocrine disruptors and additional disinfection byproducts. Also, EPA recently published lists of contaminants for which they are monitoring or seeking occurrence data.

These future rules should not affect the Logan water system because:

- Rules for contaminants like MTBE, perchlorate, atrazine, and endocrine disruptors will primarily affect unprotected supplies. Only if an upstream source of potential groundwater contamination was allowed, such as an industrial or agricultural chemical user, would these future rules become important.
- Any rule which further defines byproducts from chlorination is unlikely to have any affect on the Logan system. Although hundreds of chlorinated byproducts exist, the presence of these byproducts, as evidenced by THM concentrations, within the Logan system will be so low as to likely be non-detectable.
- The contaminants present on the newly released monitoring and potential contaminant lists are primarily trace organic chemicals. These contaminants are not expected to be found in the Logan source waters.

The distribution system rule is undefined at this time, but may have some impact on the Logan system. USEPA has identified distribution concerns to be biofilms, buried infrastructure, cross connections, treated water storage, intrusion into pipes, leaking from the pipe network, nitrification, and general water quality decay from the source water to the user. The exact nature of this rule is not clear at this time. Therefore, it is not possible to fully define the effects upon the Logan system.



7.6 Summary of Regulatory Review

The source waters supplying the City of Logan are of excellent water quality. Moreover, the DeWitt Springs source is not expected to be exposed to any future contamination, and the same can be said about the four wells as long as the City strictly enforces their source protection plans.

Even with this excellent water quality, the City should be aware of the following:

- The Consumer Confidence Report should be made available on the City website.
- Depending on how strictly the maximum contaminant level is set, some City water sources may not be able to comply with the pending Radon Rule without treatment. However, the target date for this Rule has not been set and the limits may be raised. To be better prepared, the City should sample for radon at each source quarterly for one year to document actual radon occurrence.
- The City appears to meet the 4-log virus inactivation requirements of the Ground Water Rule for users downstream of DeWitt Spring. However, the City should field verify this by sampling for free chlorine concentration at the first users.
- No water quality data was available to establish if the Crockett Well complies with the revisions to the Arsenic Rule.



8.0 Capital Improvements Plan

8.1 Capital Improvements Approach

The Capital Improvements Plan (CIP) outlines improvements that the City of Logan will need to implement over five-year periods to the year 2025.

The CIP includes a series of improvements based on the need for:

- Converting the main pressure zone into three pressure zones to resolve high pressure problems and consequent safety and maintenance issues.
- Providing redundancy and other measures to reduce system vulnerability.
- Resolving local fire flow or other delivery problems, as well as upsizing some pipe replacements based on City-identified needs.
- Expanding the service area to the west, including the large western annexation areas.
- Additional storage as required by DDW.
- Prioritized replacement of leaking pipes as explained in section 6.

The need for and development of these investment packages has been documented in section 4: Hydraulic Model Development and section 6: Pipe Replacement Plan. These are briefly described below.

8.2 Description of Capital Improvements Packages

Converting the Main Zone to Three Pressure Zones

Introducing pressure zones to reduce delivery pressures is extremely important to future operation of the water supply system. It will protect the public and reduce current high rates of pipe failures, reduce leakage, lower energy costs for well pumping, and improve water main lifetime. This effort will have to be implemented in phases.

The first phase will consist in rehabilitating and restoring to service the existing PRV stations (as well as moving one PRV station) and constructing new stations on the west side as development requires. This will resolve the dangerously high service pressures that deteriorate the network and place City staff and the public at risk.

The City has several options for achieving this objective. The most likely program, in order of implementation, is to:



1. Restore all six of the existing PRV stations to service to immediately lower supply pressures to the Central Zone. The Center Street, Crockett and 6th North wells all would continue pumping at the higher pressure of the Upper Zone and feed by gravity through the PRV stations to the Central Zone. However, the Willow Park well would be equipped with variable frequency drive and deliver to the lower pressures of the Central Zone. And the SCADA system would be upgraded as a prelude to better control of future system operations. This first step would require a minimum of capital investment.
2. In the next step construct transmission mains to allow direct delivery from the wells to the Central Zone. Reservoir capacity should also be constructed on the east and northeast sides of the Central Zone to provide operational stability.
3. In the third step, rehabilitate existing wells one at a time, and provide well pumps (with variable frequency drive) able to efficiently supply water to the Central Zone pressures, and booster pumps able to supply water to the Upper Zone.
4. Parallel to these steps, PRV stations can be established on mains west of 10th West as new developments are approved. Sharing of the cost for these stations can be negotiated with the developers.

This program will allow adjusting pressures at the PRV stations to obtain acceptable pressure delivery throughout the network, resulting in a reduction in pipe failures; reduce by more than half the energy consumption for well production; provide stable delivery; and provide crucial up-to-date operator control over system operations.

Providing Redundancy and Reducing Vulnerability

The fact that the Center Street, Crockett and 6th East wells will be able to supply water to both the Upper and Central Zones already provides a level of supply redundancy and reduces vulnerability.

This package includes first phase rehabilitation of the DeWitt Springs pipeline, and during the third investment period construction of a Ø24" steel pipeline parallel to the existing 24" concrete pipe in order to reduce vulnerability of this key line to failure.

The DeWitt Springs production facilities are also scheduled for rehabilitation to protect the source from contamination and allow higher production.



During this phase it is also anticipated to construct a new well on the east side of the Upper Zone, which will pump directly to the Golf Course Tanks, and a transmission main from the Center Street and Crockett wells. These investments will make it possible to supply the Upper Zone even if the DeWitt Springs pipeline and/or the Golf Course Tanks are out of service.

Resolving Delivery Problems

Fire flow analysis documented the need for upsizing small-diameter mains. It also confirms the need for a number of City-identified network improvements to resolve local delivery problems and accelerate the implementation of the Pipe Replacement Plan.

In several cases City staff has desired to upsize these mains in response to a combination of redundancy needs, planned development or other considerations. City policy is now to install new pipes with minimum Ø8" to replace smaller existing mains. The cost difference is not significant, and quickly resolves fire flow concerns.

This package includes replacing mains to allow better fire flow and general delivery to areas including the municipal airport, the Island, the hospital, Logan Temple, and smaller neighborhoods and larger areas on the north, west and south sides of the City.

In this package, 12" diameter mains will be installed on both sides of Main Street north of the Logan River. This will replace mains with high failure rates on the west side of Main Street, and add mains for part of the distance on the east side.

South of the Logan River, this package includes installing an Ø18" main on the west side in order to provide water to large growth areas and provide a potential emergency connection to other water utilities to the south.

This package also includes construction of two PRV stations in the Cliffside Zone to reduce extremely high pressures in two smaller neighborhoods.

Service Area Expansion

This package includes all future pipes to serve the expanded service areas on the west side of the City. The budget for these investments has been straight-line phased through the year 2025.



Additional Storage

This package includes some 8 MG of additional storage which would be required by 2025 according to current interpretation of State rules. Current recommendations by AWWA for storage requirements are considerably lower than this interpretation.

Pipe Replacement Plan

This package consists of the annual budgets from the Pipe Replacement Plan developed in section 6, applied over the four planning phases.

8.3 Capital Improvements Plan – Summary of Costs

Table 8.1 shows each of these investment packages in the Capital Improvements Plan Schedule. Details concerning each package are provided at the end of this section.

Higher unit costs have been employed for the DeWitt Pipeline investments, due to construction in difficult terrain, while lower unit costs have been applied for expansion of the service area on the rural west side. Construction costs for the DeWitt Pipeline first phase were developed in a separate rehabilitation/replacement study. Construction costs for the second phase line were estimated based on unit costs from the first study.

Table 8.1
Capital Improvements Plan – Summary of Costs

Package	Package Costs in \$million				
	2007-2010	2011-2015	2016-2020	2021-2025	Total Cost
1. Convert Main Zone	\$1.60	\$5.84	-	-	\$7.44
2. Provide Redundancy	\$8.29	\$3.29	\$8.58	-	\$20.16
3. Resolve Delivery Problems	\$3.24	\$6.05	\$1.69	-	\$10.97
4. Service Area Expansion	\$7.32	\$9.15	\$9.15	\$9.15	\$34.76
5. Additional Storage	-	-	-	\$13.16	\$13.16
6. Pipe Replacement Plan	\$3.50	\$4.38	\$4.38	\$3.51	\$15.77
Total	\$23.94	\$28.70	\$23.79	\$25.81	\$102.25
Annual Cost each period	\$5.99	\$5.74	\$4.76	\$5.16	

It is worth noting the following:

- Converting the main zone to three pressure zones represents only about 7% of the total CIP cost over the planning period.



- Providing redundancy, and resolving fire flow and other delivery problems represent about 30% of the total CIP investment – where the DeWitt Springs pipeline and springs rehabilitation represent about 14% of the total CIP investment.
- Expanding the service area in response to growth represents about 34% of the total CIP investment.
- Providing additional storage required by Utah DDW represents about 13% of the total CIP investment.
- Annual pipe replacement represents 14-18% of the annual CIP investment budgets.

8.4 Capital Improvements Plan – Detailed List of Costs

Estimates were based on

1. Construction costs increased by 10% for mobilization, insurance and permitting;
2. This amount increased by 30% for contingencies;
3. This amount increased by 15% for professional services (surveying, engineering, construction supervision).

Table 8.2 provides a detailed listing of investments under each of the six categories shown in Table 8.1.

Table 8.2					
Package	Package Costs in \$million				
	2007-2010	2011-2015	2016-2020	2021-2025	Total Cost
1. Convert Main Zone	\$1.601	\$5.836	-	-	\$7.437
SCADA system upgrade	\$0.411				
PRV stations, rehabilitation	\$0.082				
PRV station, new near Center St.	\$0.082				
Crockett mains, 16"	\$0.192	\$0.240			
6 th East-NE Tank mains, 16"	\$0.423	\$0.528			
Well screens		\$0.066			
Well pumps, VFDs		\$0.658			
Generator, one well		\$0.033			
Booster pumps		\$0.148			
PRV stations, Lower Zone		\$0.411			
SE Tank (elevated)		\$0.822			
SE Tank main		\$0.052			
2. Provide Redundancy	\$8.290	\$3.289	\$8.575	-	\$20.155
DeWitt Springs rehabilitation	\$1.069				
DeWitt pipeline (option 3A)	\$6.868				



Table 8.2

Capital Improvements Plan – Detailed List of Costs

Package	Package Costs in \$million				
	2007-2010	2011-2015	2016-2020	2021-2025	Total Cost
Main to hospital, 16"	\$0.354	\$3.289			
New well, 5 mgd capacity			\$0.793		
Main to Golf Course tanks from wells -			\$0.843		
- 18" main from Center Street			\$1.076		
- 24" main from Crockett			\$5.863		
- 30" main from new well					
DeWitt pipeline (2 nd phase)					
3. Resolve Delivery Problems	\$3.237	\$6.045	\$1.687	-	\$10.969
Airport supply, 14"	\$0.345				
5 th East to 8 th East, 8"	\$0.088				
Hospital & 6 th East, 16"	\$0.262				
Hospital, southwest area, 8"	\$0.289				
2 nd East, 10 th to 11 th North, 10"	\$0.019				
Dugway, 24"	\$0.083				
Temple, northeast diagonal, 8"	\$0.023				
2 nd East, 2 nd North to 2 nd South, 8"	\$0.073				
1 st East, 1 st North to 650 South, 10"	\$0.241				
3 rd South and 5 th East, 8"	\$0.063				
4 th North, Main Street to 1 st East, 8"	\$0.068				
6 th North, 150 West to 10 th West, 8"	\$0.934				
Palatial Living, 8"	\$0.318				
2 nd North, Main St. to 5 th West, 10"	\$0.080				
Cliffside PRV stations, two new	\$0.164				
10 th West, 18 th North to Airport, 18"	\$0.189	\$0.162			
Island supply, 18"		\$0.405			
Canyon-Sumac, 8"		\$0.407			
Sumac, 8"		\$0.318			
Temple, east area, 8"		\$0.022			
3 rd East, 2 nd to 4 th North, 8"		\$0.058			
Main St. north of river, west side 12"		\$0.927			
Main St. north of river, east side 12"		\$1.557			
Main St. south of river, west side, 18"		\$1.797			
18 th South to mobile homes, 12"		\$0.392			
14 th North, 6 th East to 2 nd West, 10"			\$1.687		
4. Service Area Expansion	\$7.318	\$9.148	\$9.148	\$9.148	\$34.762
18" on 18 th South	\$0.024	\$0.030	\$0.030	\$0.030	
12" lines	\$7.294	\$9.117	\$9.117	\$9.117	
5. Additional Storage (DDW required)	-	-	-	\$13.156	\$13.156
4 MG new				\$6.578	
4 MG new				\$6.578	
6. Pipe Replacement Plan	\$3.496	\$4.383	\$4.382	\$3.505	\$15.767



Package	Package Costs in \$million				
	2007- 2010	2011- 2015	2016- 2020	2021- 2025	Total Cost
years 1-4	\$3.496				
year 5		\$0.874			
years 6-9		\$3.509			
year 10			\$0.877		
years 11-14			\$3.505		
years 15-19				\$3.505	
Total	\$23.944	\$28.701	\$23.793	\$25.809	\$102.246
Annual Cost each period	\$5.986	\$5.740	\$4.759	\$5.162	