Lincoln Water System Facilities Master Plan

CITY OF LINCOLN NEBRASKA

City of Lincoln Project No. 701353

May 2014

HDR Project No. 214269

HDR

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HDR ONE COMPANY | *Many Solutions*

May 12, 2014

Steve Owen
Superintendent of Water Production and Treatment
Lincoln Water System
2021 North 27th Street
Lincoln, Nebraska 68503

RE: 2013 Facilities Master Plan

Dear Mr. Owen:

HDR Engineering, Inc. is pleased to submit the 2013 Facilities Master Plan for the Lincoln Water System. This plan is a comprehensive master plan that addresses future water supply, treatment and distribution system infrastructure needs for the Lincoln Water System through the year 2060. The planning horizons used in the Master Plan have been coordinated with the Lincoln/Lancaster County 2040 Comprehensive Plan (LPlan 2040).

We appreciate the efforts of the Lincoln Water System staff and other City departments on the development of this Master Plan. These efforts were instrumental in the development of the Master Plan which will serve as a roadmap for Lincoln Water System.

Thank you for the opportunity to work on this exciting project and we look forward to the opportunity to work with you again in the future.

Sincerely,

HDR ENGINEERING, INC.

J Erin Hunt

J. Erin Hunt, PE

Project Manager

Enclosure

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Acknowledgements

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CITY OF LINCOLN NEBRASKA

Lincoln Water System Facilities Master Plan

Executive Summary

May 2014

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Abbreviations and Acronyms

ADD	Average Day Demand
AMD	Average Month Demand
AWWA	American Water Works Association
CIP	Capital Improvement Program
City	City of Lincoln
cfs	cubic feet per second
CMMS	Computerized Maintenance Management System
CPI	Consumer Price Index
Cr-6	hexavalent chromium
cVOC	carcinogenic Volatile Organic Compounds
DSC	Debt Service Coverage
EPA	U.S. Environmental Protection Agency
FY	Fiscal Year – September 1st to August 31st
GIS	Geographic Information System
HCW	Horizontal Collector Well
HDR	HDR Engineering, Inc.
IT	Information Technology
LWS	Lincoln Water System
2007 Master Plan	2007 Facilities Master Plan
Master Plan	2013 Facilities Master Plan
LPlan 2040	Lincoln/Lancaster County 2040 Comprehensive Plan
MDD	Maximum Day Demand
mgd	million gallons per day
MHD	Maximum Hour Demand
MMD	Minimum Month Demand
ng/L	nanograms per liter
NPDES	National Pollutant Discharge Elimination System
NPV	Net Present Value
SDWA	Safe Drinking Water Act

SP	Seasonal Peak
UCMR	Uncontaminated Monitoring Rule
UCMR3	Third Uncontaminated Monitoring Rule
VOC	Volatile Organic Compound

1.0 Introduction

Water utilities must continuously plan to address system needs and challenges, such as system growth, aging infrastructure, increasingly stringent regulatory requirements, and the need for a well-planned and efficient Capital Improvement Program (CIP). Recognizing this need, the City of Lincoln (City) has historically conducted master planning efforts at 5-year intervals: a comprehensive master planning effort every 10 years and updates to address system growth and distribution system needs every 5 years. The City last completed a comprehensive Facilities Master Plan in 2002 and an update in 2007.

The 2013 Facilities Master Plan (Master Plan) will provide a guide for the short-term and long-term improvements for the infrastructure of the Lincoln Water System (LWS) through the year 2060. The anticipated growth of the system through this time period was coordinated with the Lincoln/Lancaster County 2040 Comprehensive Plan (LPlan 2040). Figure ES-1 presents the study area and anticipated growth tiers for this planning effort.

The Master Plan presents recommended improvements for the City's water supply, treatment, transmission, and distribution facilities based on projected future water capacity requirements and the need for renewal and replacement in the system. The recommended improvements presented in the Master Plan will be the basis for financing, design, and construction of future water infrastructure needs. HDR Engineering, Inc. (HDR) and City staff have worked together extensively throughout this planning process to ensure all aspects of the City's water planning needs have been met.

2.0 Water Capacity Requirements

2.1 Population

Historical population data for the City was obtained from the LWS 2007 Facilities Master Plan (2007 Master Plan) and the U.S. Census Bureau. The 2012 base year population was estimated from the 2010 population based on an annual growth rate of 1.2 percent. Future population projections were based on the LPlan 2040. In addition to an overall population growth projection, the population projections were distributed geographically to provide more detail for future improvements. A summary of historical and projected population is presented in Figure ES-2.

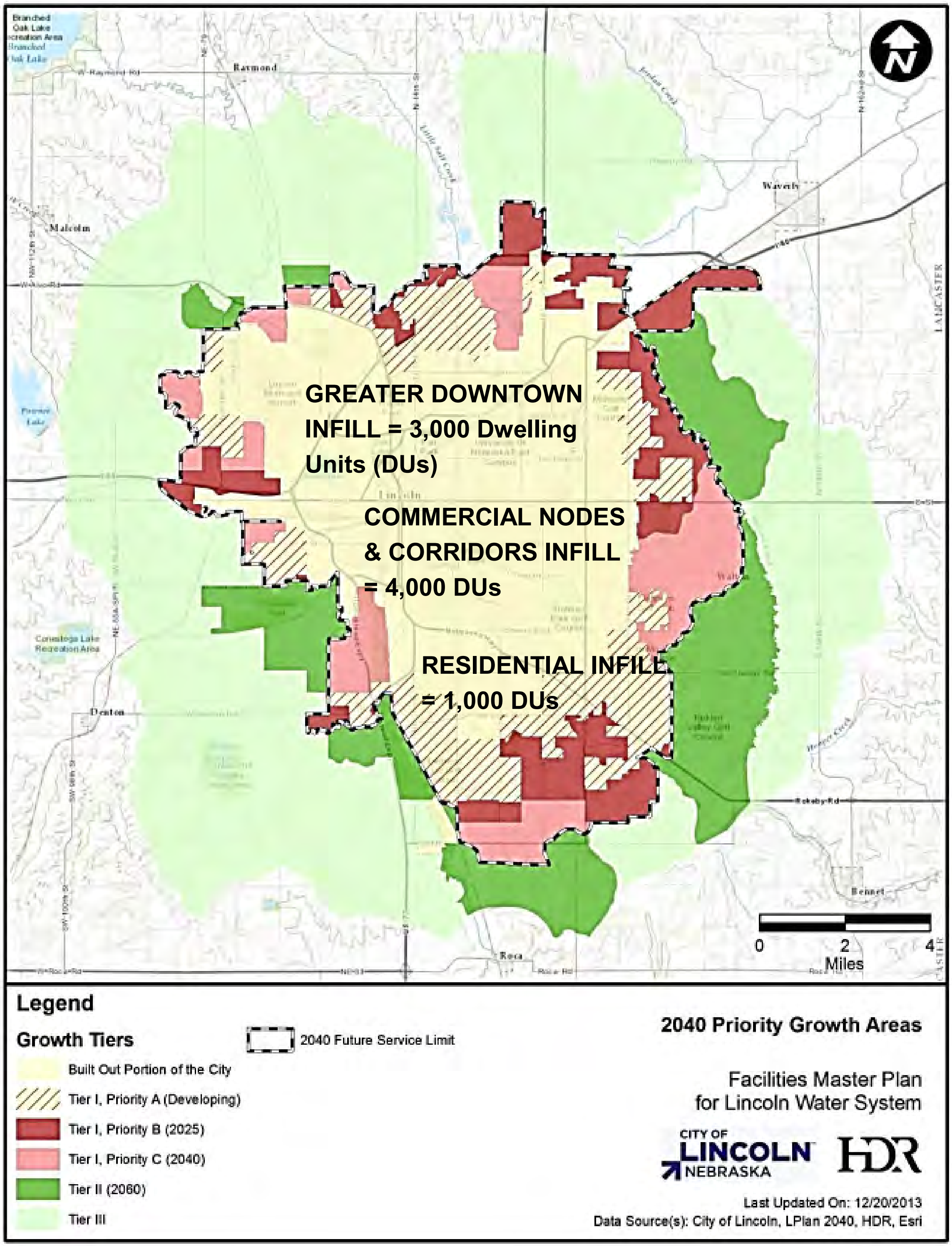

Figure ES-1 Study Area

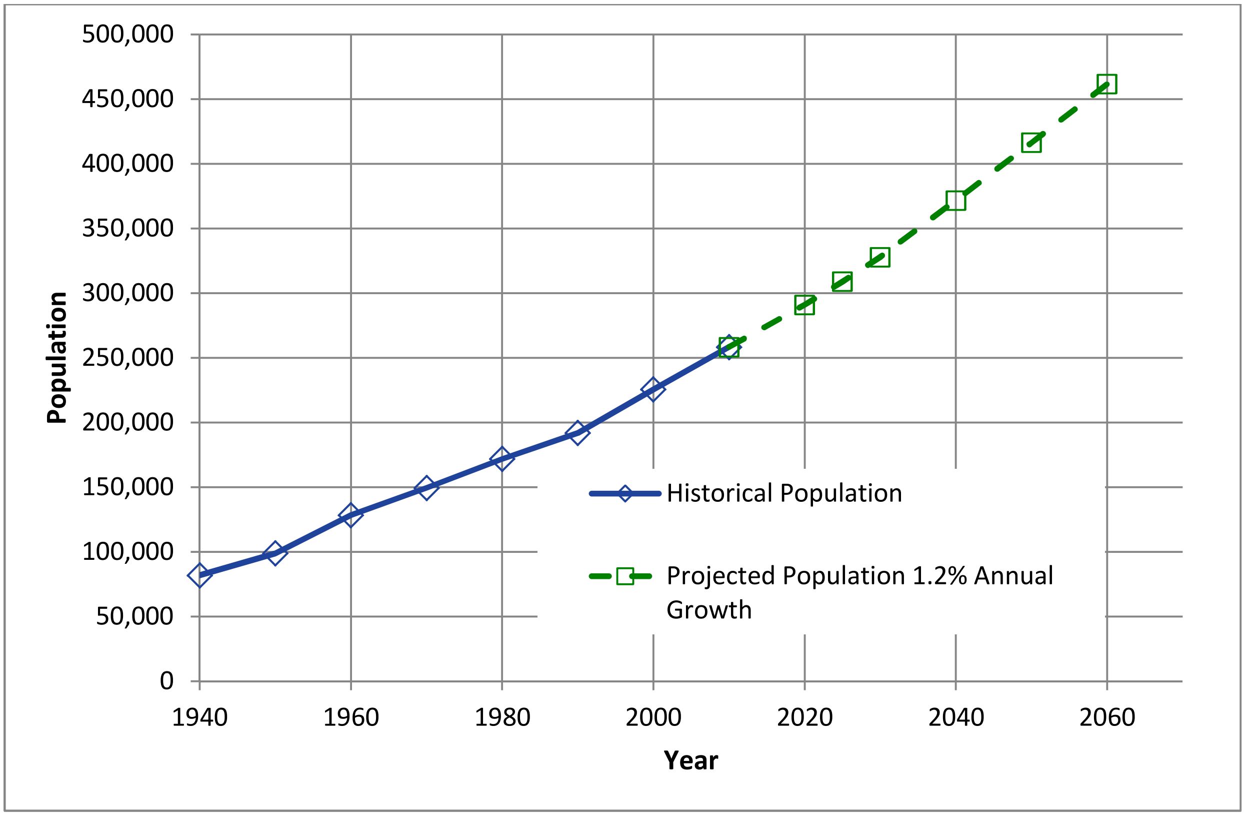

Figure ES-2 Historical and Projected City of Lincoln Population

2.2 Water Capacity

Future water capacity requirements were determined based on projected population and per capita water usage. Due to the City's promotion of water conservation, the per capita water usage rates have declined since the 2007 Master Plan, and this decline is reflected in the future capacity projections. Over the past 20 years, the per capita water usage rates have declined by approximately 23 percent. A summary of the future water capacity requirements is presented in Figure ES-3. Some of the key terms used in the capacity and supply analysis are defined as follows:

- Well field pumpage is the amount of water delivered to the Platte River Water Treatment Facility from the well field.
- Lincoln usage is the amount of water transmitted to the distribution system from the Platte River Water Treatment Facility.
- Average Day Demand (ADD) is the total water used during the year divided by 365 days per year.

- Maximum Day Demand (MDD) is the total daily demand during the day with the greatest amount of demand in a given year.
- Maximum Hour Demand (MHD) is the water demand during the hour with the highest system demands in a given year.
- Seasonal Peak (SP) is the average demand over the highest consecutive 3 months of raw water supply.

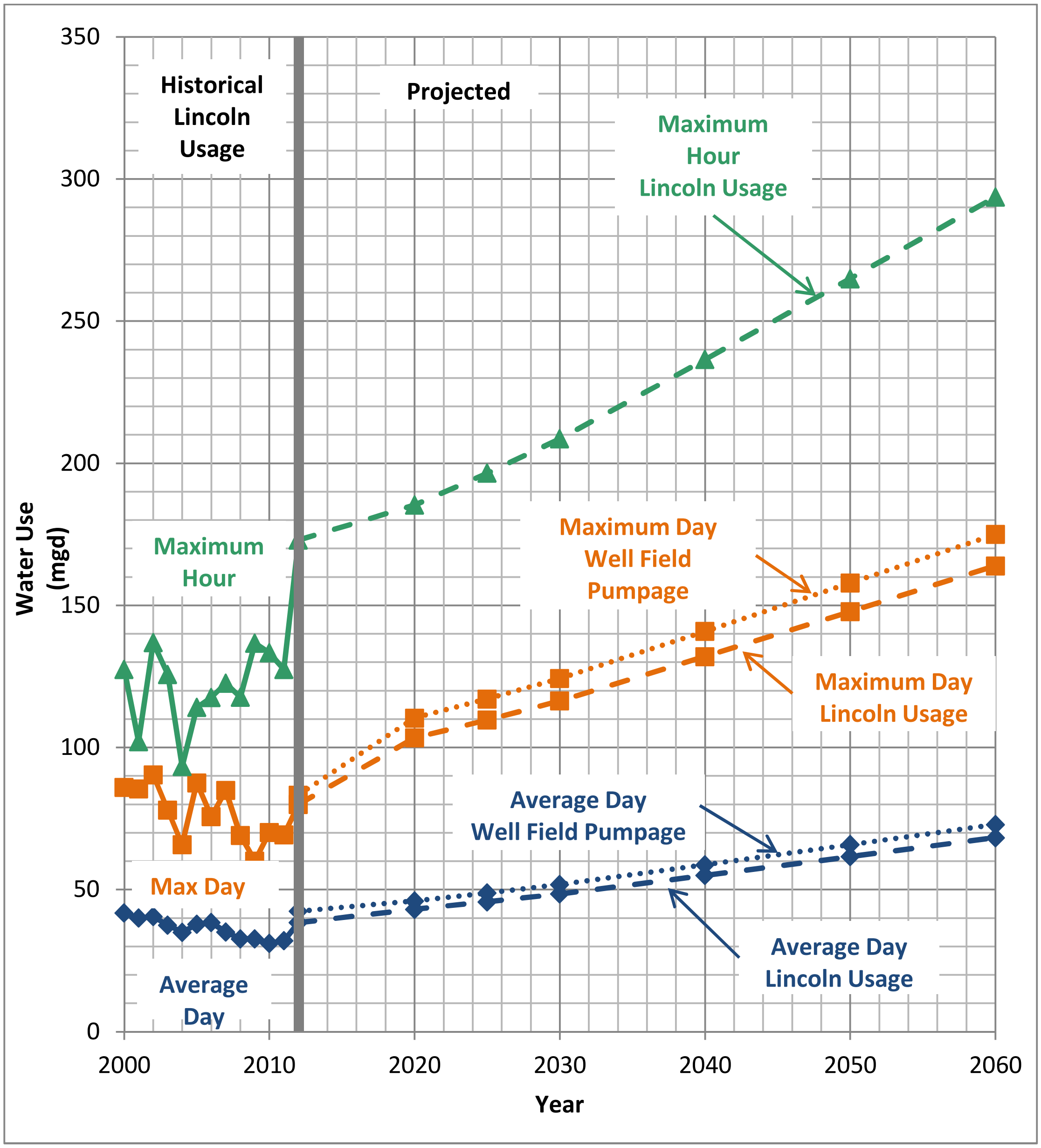

Figure ES-3 Future Demand Projections

3.0 Water Supply

The Master Plan establishes the current capacity of the City's water supply and identifies alternatives to expand those supplies through the year 2060. The planning effort includes a review of hydrologic data to determine the reoccurrence interval of prolonged drought events, an evaluation of current well performance data to determine the seasonal firm capacity of the well field, a comparison of the firm capacity of the well field to projected future water demands, and an evaluation of alternatives to add sufficient supply to account for any projected water deficit. The firm capacity of the well field is the capacity of the well field with the largest well out of service. The seasonal firm capacity is the capacity of the well field with the largest well out of service during the summer months.

3.1 Source Water Availability

A hydrologic analysis was performed to evaluate streamflow conditions in the Platte River. The long-term yield of the City's raw water supply is correlated to the streamflow in the Platte River; therefore, understanding the flow regime of the river is an important part of the Master Plan effort. The objective of this analysis is to determine reoccurrence intervals (or frequency) of prolonged droughts and to understand the duration of these events.

The drought experienced by the City during summer 2012 was a 50- to 100-year reoccurrence interval event for the 7- and 30-day duration events, and a 50-year reoccurrence interval event for the 60-day duration event. With a 50-year planning horizon and the reoccurrence interval of the 2012 event being approximately 50 years, there is a strong probability (64 percent) that the City will experience at least one drought event similar to the 2012 event during the planning horizon.

Based on the results of the analysis, it is recommended that the availability of the water supply be evaluated over a range of streamflow values bracketed by the 50-year, 60-day and 100-year, 30-day events. This equates to Platte River flows at Ashland, Nebraska, of 466 cubic feet per second (cfs) and 311 cfs, respectively.

3.2 Supply Analysis

3.2.1 Well Field Pumping Capacity

The City's well field consists of 40 vertical wells, 2 existing horizontal collector wells (HCWs), and a third HCW that is currently under construction. When construction of the third HCW is complete, the City's well field will have a total pumping capacity of 192 million gallons per day (mgd) and a firm pumping capacity of approximately 149 mgd.

The analysis of the supply capacity of the existing well field indicated that once construction of the third HCW is complete, the well field will have a maximum instantaneous capacity of between 110 and 130 mgd, depending on streamflow conditions. The summer seasonal capacity of the well field for 60- to 90-day production capacity ranges from 75 to 80 mgd when streamflow is less than 1,000 cfs and from 71 to 77 mgd during a streamflow event that correlates to a 100-year reoccurrence interval drought for the same duration. Table ES-1 presents a summary of these capacities, and Figure ES-4 presents well field capacities relative to projected demands.

Table ES-1 City's Well Field Seasonal Firm Capacity with the third HCW

Season	Streamflow (cfs)	Water Level Conditions	Maximum Instantaneous Pumping (mgd)	Maximum Pumping for 2 Months (mgd)	Maximum Pumping for 3 Months (mgd)
Spring	3,000 or greater	High	130	110	105
Summer low flow	1,000 to 500	Low	120	80	75
Summer drought	<500 cfs 100-Year Reoccurrence Interval	Low	110	77	71

Note:

The 100–year, 60-day average streamflow during drought at Ashland is 351 cfs, and the 100-year, 90-day average streamflow during drought at Ashland is 465 cfs.

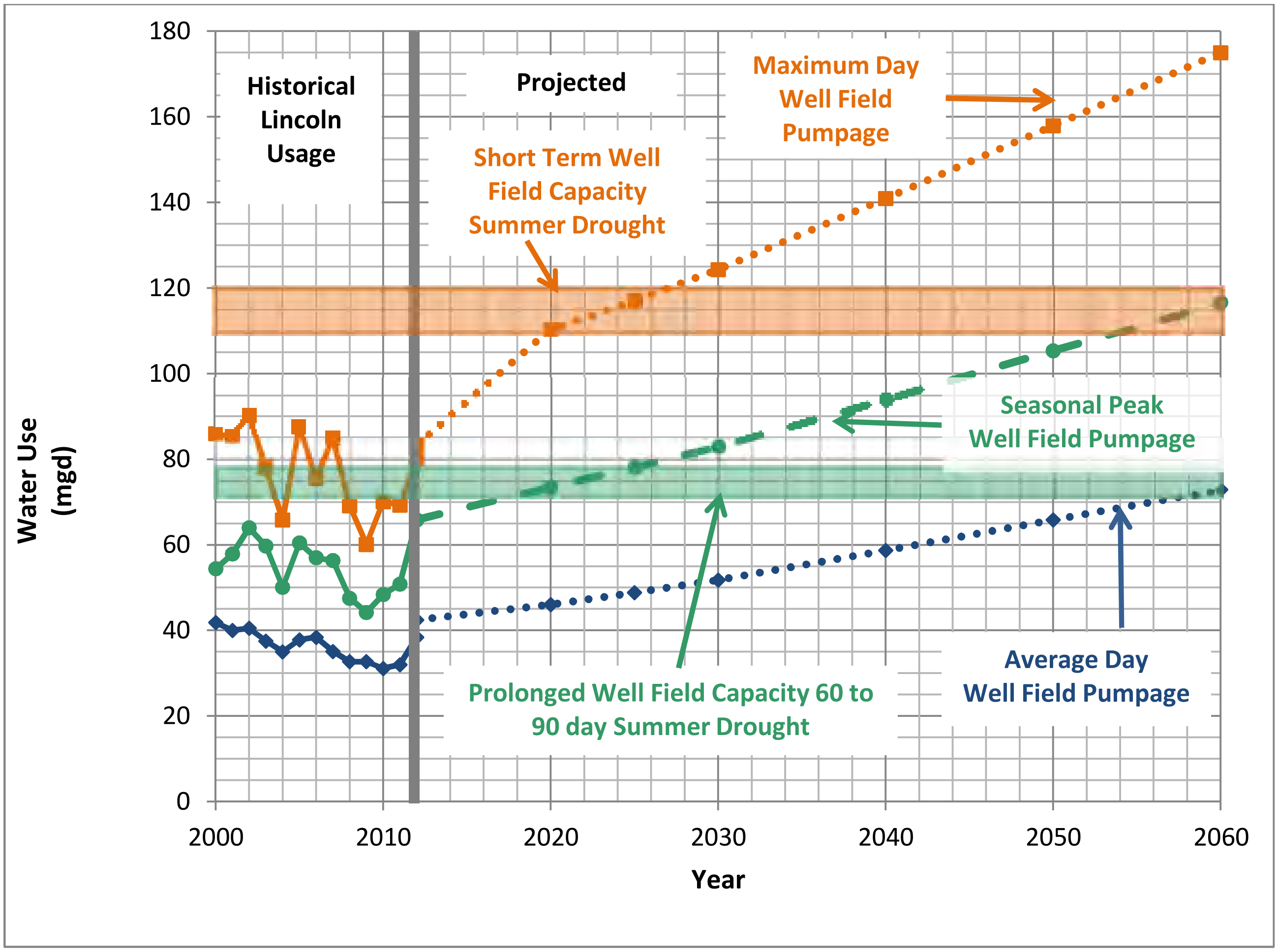

Figure ES-4 Raw Water Supply Deficits over Planning Horizon (with the third HCW)

3.2.2 Alternative Supply Analysis

The City's well field, in its current configuration, will not be able to meet projected demands through the planning horizon of 2060. Therefore, an evaluation was conducted of raw water supply alternatives that would increase the raw water capacity of the City to meet both the short-term and long-term demands. This analysis also considered increasing the reliability of the raw water supply by diversifying the raw water source. Three planning horizons were identified for this evaluation, as defined below.

Short-term horizon (2014-2025)

In the short-term horizon (2014-2025), the projected raw water demand could exceed the 60- to 90-day pumping capacity as early as 2018 depending on the magnitude and duration of a drought. The instantaneous and short-term pumping capacity could be exceeded by 2022. These short-term supply alternatives should be able to increase the instantaneous and short-term water supply capacity by 20 mgd and should be able to increase the summer seasonal yield by 10 mgd. Because this short-term supply alternative must be developed in the near future, the alternative must be considered relatively easy to permit and construct.

Short-term supply alternatives evaluated in the Master Plan include:

- Expansion of existing well field with completion of a fourth HCW
- New well field in the High Plains Aquifer
- Aquifer storage and recovery as peak shaving
- Metropolitan Utilities District (MUD) interconnection
- Water reuse

The recommended option is expansion of the existing well field with the construction of the fourth HCW. The opinion of cost for this option is $10.3 million in 2013 dollars. The caisson for the fourth HCW is expected to be completed in 2014. This alternative would include the completion of this well for production and the construction of the other related components required to connect the well to the system.

Mid-term horizon (2026-2040)

In the mid-term horizon (2026-2040), raw water demands are projected to exceed the supply capacity by 15 to 25 mgd by 2040 during periods of prolonged drought. The mid-term supply alternatives evaluated in the Master Plan include:

- Expansion of the existing well field
- Development of a surface water reservoir

The recommended option is expansion of the existing well field. Through a separate contract with the City, groundwater model simulations were performed that considered a well field consisting of 40 vertical wells and 6 HCWs. Under this well field configuration, the model estimated that the well field could produce 111 mgd for 60 days and 107 mgd for 90 days with a streamflow of 200 cfs. These modeled values compare favorably to the values estimated using conservative summer HCW pumping rates of 10 mgd for the new HCWs.

Assuming each new HCW will increase the summer seasonal well field yield by 10 mgd and the maximum instantaneous pumping rate by 15 mgd, a fifth HCW would increase the summer seasonal pumping capacity of the well field to between 91 and 97 mgd during drought conditions. This pumping capacity would meet projected seasonal demands through 2035. The addition of a sixth HCW on the East Bank of the well field would further increase the summer seasonal pumping capacity of the well field to between 101 and 107 mgd during drought conditions. This pumping capacity would meet projected seasonal demands until approximately 2045.

The opinion of cost for the fifth HCW is $12.6 million in 2013 dollars. The construction of the sixth HCW will require an additional 48-inch raw water transmission main to convey the water from the well to the Platte River Water Treatment Facility. The opinion of cost for the sixth HCW and the transmission main is $24.3 million in 2013 dollars.

Long-term (2041-2060) horizon

The long-term (2041-2060) supply alternatives evaluated included the Missouri River and the Platte River alluvial aquifers. The Missouri River was selected as the preferred alternative. The Missouri River is operated as a navigable channel, and the streamflow is regulated from upstream reservoirs. A well field constructed in the Missouri River alluvium would be less susceptible to low streamflow during the summer months when demands for water are highest.

For the purposes of the Master Plan, it was assumed that the long-term alternative would supply a maximum of 60 mgd, which is sufficient to meet the water supply needs of the City through 2060 if the mid-term supply alternative is not developed. The implementation of the mid-term supply option would reduce the initial capacity needs for the long-term alternative. However, development of a 60-mgd supply along the Missouri River would provide the City with a diversified source of supply that is more resistant to drought and could provide opportunities to develop this supply option as a regional water supply.

The opinion of cost for this alternative is approximately $500 million in 2013 dollars and includes the well field, a treatment facility, and transmission mains necessary to connect the Missouri River Project to the distribution system. It is recommended that field investigation for well field site selection be conducted in 2016 and that land acquisition for the well field facility occur in approximately 2018 in order to secure a site for future source development.

3.3 Water Conservation

It is the policy of the City to promote water conservation. Water conservation encourages responsible use and preservation of the City's water supply. While conservation has not been considered as a source of supply in this Master Plan, it could be used as a means to further delay the need for expanding the City's existing water supply.

The City has recently updated its Water Management Plan. When water use cannot be maintained within the system's capacity, the plan defines procedures and provides guidance for imposing water restrictions. The City also maintains the Mayor's Water Conservation Task Force, which is composed of community members appointed by the Mayor. The focus of the Mayor's Water Conservation Task Force is to promote voluntary cooperation to accomplish conservation goals.

In addition to the demand management, tiered rate structure, education, and public information efforts the City has already implemented, other potential water conservation practices may be feasible, including:

- Customer water survey and audit programs
- System water audits, leak detection, and leak repair
- Large landscaping conservation programs and incentives
- Incentive programs for water-efficient fixtures and appliances
- Conservation programs for commercial, industrial, and institutional accounts

4.0 Water Treatment

Water is treated at the Platte River Water Treatment Facility. There are two treatment trains at this site: the East Plant and the West Plant. The East Plant has a capacity of 60 mgd and primarily treats water from the HCWs. The East Plant consists of oxidation for iron and manganese removal, filtration, disinfection, and fluoridation. The East Plant has been designed for four future expansions in 30 mgd increments.

The West Plant has a capacity of 60 mgd and treats water from the vertical wells. The West Plant consists of aeration, chlorine oxidation of manganese, filtration, disinfection, and fluoridation.

4.1 Water Quality and Regulatory Requirements

Drinking water standards are regulated by the U.S. Environmental Protection Agency (EPA) under the Safe Drinking Water Act (SDWA). Public water supplies must follow the standards set forth by EPA. A review of raw water quality and finished water quality shows that LWS is in full compliance with the current drinking water standards.

A review of anticipated future regulations is included to assist in planning for future treatment improvement needs. Likely regulatory actions occurring in the 2014-2015 time frame will come from the preliminary Third Regulatory Determination, the proposed Long-Term Lead and Copper Rule, the proposed carcinogenic Volatile Organic Compounds Rule (cVOC), or the proposed Perchlorate Rule. Actions further out in time will arise from the third 6-year review process or from separate actions directed by legislation.

The following are the potential regulations and their impacts on LWS:

- Nitrosamines
  - Current international guidelines range from 40 to 100 nanograms per liter (ng/L)
  - Maximum found in LWS sampling was 2.8 ng/L (2008-2009)
  - Potential impacts on LWS are negligible
- Proposed Perchlorate Rule
  - Potential regulatory level ranges from 2 to 10 μg/L

  - LWS found no perchlorate during uncontaminated monitoring rule (UCMR) sampling
  - Potential impacts on LWS are negligible
- Proposed Long-Term Lead and Copper Rule
  - Partial lead service line replacement
  - Sample site modifications
  - Potential impact on LWS will be altered sampling procedures
- Proposed cVOC Rule
  - None of the proposed compounds have been detected in LWS samples
  - Potential impacts on LWS are negligible

The SDWA requires EPA to review all drinking water regulations every 6 years for possible revision. Expectations are that the following rules will be included for revision: the Stage 2 Disinfection Byproduct Rules, the Interim Enhanced Surface Water Treatment Rule, and the Long-Term 2 Enhanced Surface Water Treatment Rules. In addition, chromium is likely to be included as part of the 6-year review list because hexavalent chromium (Cr-6) is of concern.

The list of contaminants on the third uncontaminated monitoring rule (UCMR3) program provides insight into which compounds might be further regulated in the future. That list includes a few metals and some Volatile Organic Compounds (VOCs), several perfluorocarbons, 1,4-dioxane, two viruses, and seven hormones, along with total chromium and hexavalent chromium. LWS will be sampling for UCMR3 in March, June, September, and December 2015.

4.2 Water Treatment Plant Improvements

4.2.1 Future Capacity Expansions

Water treatment plant improvements and expansions will be required as system demands increase. Table ES-2 lists the projected improvements, the timing of implementation, and estimated costs of the recommended improvements.

Table ES-2 Future Plant Improvements

Year[1]	Description	Current Cost Basis[2]	Future Cost Basis - 3% Inflation[3]	Future Cost Basis - 5% Inflation[4]
2027	12 mgd West Plant Expansion[5]	$14,588,000	$22,043,000	$28,827,000
2034	First 30 mgd East Plant Expansion	$25,200,000	$46,872,000	$70,207,000
2052	Second 30 mgd East Plant Expansion[6]	$23,800,000	$75,589,000	$159,579,000

Notes:
1. *The year listed is when the additional capacity is operational.*
2. *Current cost based on HDR Project Cost Estimating software, 2013 dollars.*
3. *Inflated to projected year dollars at 3% per year inflation rate.*
4. *Inflated to projected year dollars at 5% per year inflation rate.*
5. *Testing for the West Plant Expansion is projected to be completed in 2022.*
6. *The need for the second East Plant expansion is dependent on the final timing of implementation of the Missouri River Project.*

4.2.2 Regulatory Improvements

Both East and West Plants are in compliance with all existing drinking water regulations. Rulemaking in the foreseeable future does not appear to adversely impact the plants. Only National Pollutant Discharge Elimination System (NPDES) discharge permit improvements are anticipated to possibly be required throughout the planning period.

5.0 Transmission and Distribution Systems

The transmission and distribution system analysis focused on maintaining and expanding service to customers as the City grows over the next 50 years. The planning periods were selected to coordinate with the LPlan 2040 and are as follows:

- 2025 (short-term, 2014-2025)
- 2040 (mid-term, 2026-2040)
- 2060 (long-term, 2041-2060)

5.1 Existing Facilities

LWS facilities are categorized into two major systems: transmission and distribution. The transmission system consists of transmission mains, reservoirs, and pumping stations that deliver water from the Platte River Water Treatment Facility near Ashland to the distribution system within the City. The distribution system consists of distribution mains, reservoirs, and pumping stations that deliver water from the transmission system to LWS's customers. The distribution system is divided into six pressure zones or service levels: Northwest, Belmont, Low, High, Southeast, and Cheney. The service levels are shown in Figure ES-5.

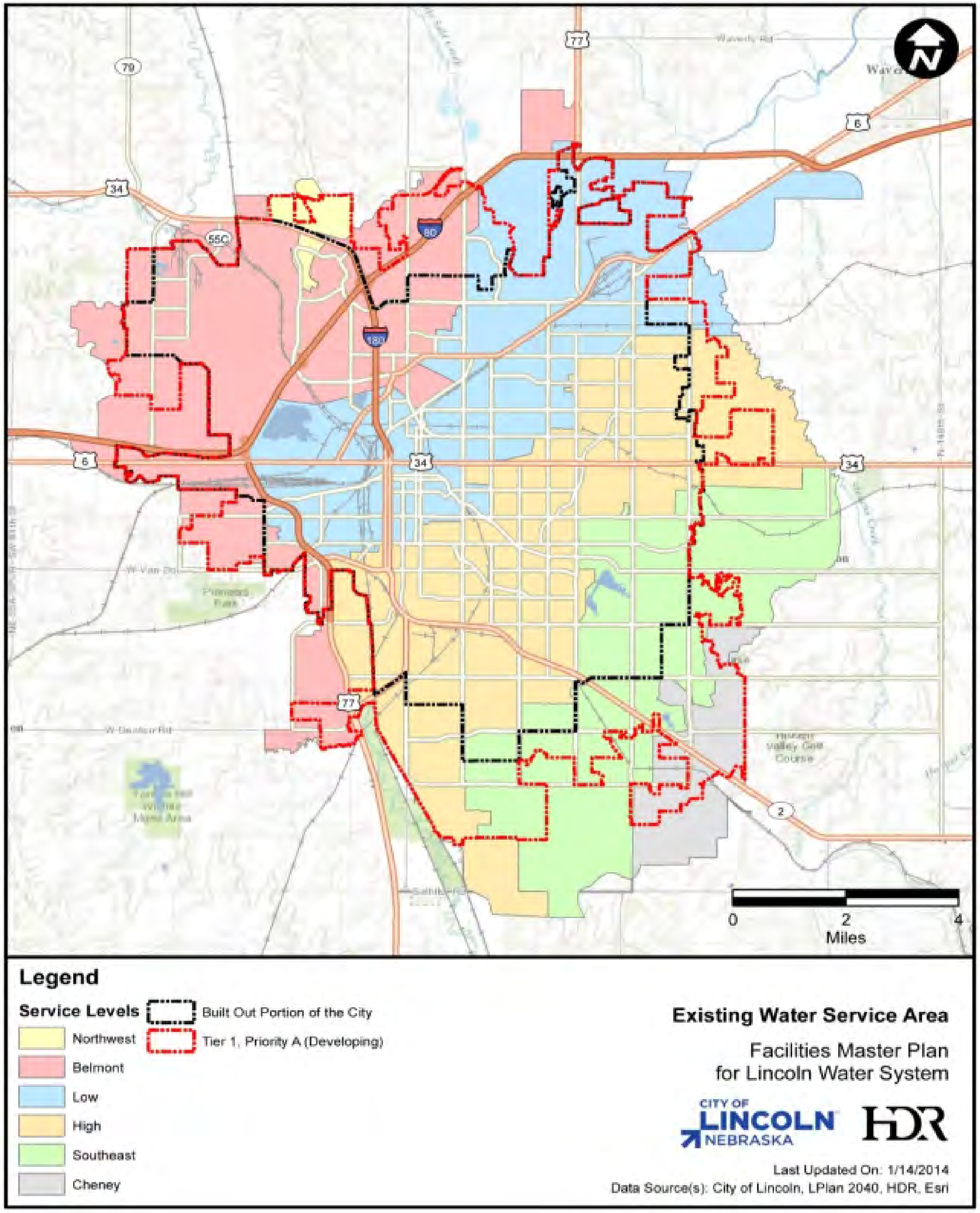

Figure ES-5 Existing Water Service Area

5.2 Analyses

A computer hydraulic model was used to analyze the transmission and distribution systems for each of the planning periods. Computer hydraulic analysis is a method of predicting hydraulic gradients, pressures, and flows across the water distribution network under a given set of conditions. The model software used is InfoWater by Innovyze, version 10.0 Update 7. Alternative improvements were investigated to identify those most effective in meeting future system needs. Criteria used to develop the improvement program include increasing system reliability, simplifying system operations, effectively utilizing system storage, and maintaining minimum pressures under maximum hour demand conditions.

5.2.1 Fire Flow Analyses

The base year maximum day demands were used to analyze potential fire flow deficiencies in the distribution system. Zoning-based fire flow requirements were established as a general indication of areas of potential deficiencies. Some deficiencies were found in areas of the system with 4-inch mains and 6-inch non-looped mains in older areas of the City, including the downtown area. The results of the analyses were incorporated into the development of the improvements program.

5.2.2 Water Age Analyses

Water age modeling was performed to identify areas in the distribution system with high residence times. It is acceptable industry practice to use distribution system water ages as a surrogate indicator for many water quality parameters, including disinfection by-product formation, disinfectant decay, corrosion control effectiveness, microbial re-growth, nitrification, and taste and odor issues. Water age should not be considered as the ultimate indicator of these aforementioned water quality characteristics, but in conjunction with other factors such as pipe characteristics, disinfection processes, distribution system operations, and water use habits. However, water age can be quite useful in identifying distribution system deficiencies in terms of water quality.

Each water age scenario was simulated for a duration of 30 days (720 hours). Areas of the system fed by the Air Park, Southeast, and Cheney tanks have the highest water age in the system. The overall system water age is 148 hours and 103 hours during minimum month demand (MMD) and average month demand (AMD), respectively.

5.3 Recommended Improvements

Improvements are recommended to be completed in four phases: Immediate (2014-2019), Short-term (2020-2025), Mid-term, (2026-2040), and Long-term (2041-2060). The improvements are divided into two categories: transmission improvements and distribution improvements. A summary of the total cost for each phase of improvements is provided in Table ES-3.

Table ES-3 Summary of Transmission and Distribution Improvements

Description	Current Cost Basis[1]	Future Cost Basis (3%)[2]	Future Cost Basis (5%)[3]
Immediate (2014-2019)			
Transmission Improvements	$12,186,000	$14,141,000	$15,584,000
Distribution Improvements	$18,459,000	$20,233,000	$21,501,000
Short-term (2020 -2025)			
Transmission Improvements	$46,427,000	$60,759,000	$72,449,000
Distribution Improvements	$25,442,000	$33,731,000	$40,519,000
Mid-term (2026 -2040)			
Transmission Improvements	$106,041,000	$183,747,000	$264,180,000
Distribution Improvements	$31,570,000	$57,427,000	$84,772,000
Long-term (2041-2060)			
Transmission Improvements	$92,356,000	$239,688,000	$451,520,000
Distribution Improvements	$70,710,000	$211,086,000	$430,017,000

Notes:
1. *Engineering and Contingency estimates are included in each item at a value of Contingency 30% and Engineering 20% of the item cost.*
2. *Inflated to projected implementation year dollars at 3% per year inflation rate.*
3. *Inflated to projected implementation year dollars at 5% per year inflation rate.*

6.0 Water Main Replacement Program

The LWS distribution system consists of a wide range of pipe sizes, ages, and materials. As of the end of 2012, there were approximately 1,200 miles of water main ranging in size from 4 to 60 inches. The oldest pipes in the system were installed in the late 1800s. Over the past 29 years, LWS has added an average of 16 miles of water main per year.

Currently, LWS has budgeted $4.0 million for main replacements in fiscal year (FY) 2013. This will replace approximately 5 miles, or 0.4 percent, of the overall distribution system. This budgeted replacement rate is expected to continue over the next several years.

LWS uses an asset ranking form to prioritize potential projects based on several criteria, including:

- Level of service consequence
- Damage consequence

- Water main condition and failure risk

The score from this asset ranking is combined with other factors such as:

- Break history
- Capacity improvements
- Fire flow improvements
- Opportunity projects (replacing water mains coincident with roadway projects)

LWS has set a system goal to have a maximum break rate of 14 breaks per 100 miles of main per year. Overall, LWS has maintained break rates at or near the goal. LWS is currently performing better than the national average as reported by American Water Works Association (AWWA). Figure ES-6 presents main break rate history for LWS. It is estimated that the current replacement program has prevented nearly 1,400 additional breaks since 1991.

The break rates related to various pipe characteristics were compared to determine where trends exist in order to assist in prioritizing projects. The pipe traits that were used in the comparison are:

- Pipe material
- Pipe diameter
- Operating pressure
- Ground slope
- Soil corrosion potential

The break rates versus age for these criteria were compared to determine if there were significant differences in the break rate. It was found that diameter and material showed the most significant variation in break rates; 6-inch and smaller diameters and thin walled cast iron and unprotected ductile iron had higher break rates than other categories in the comparison.

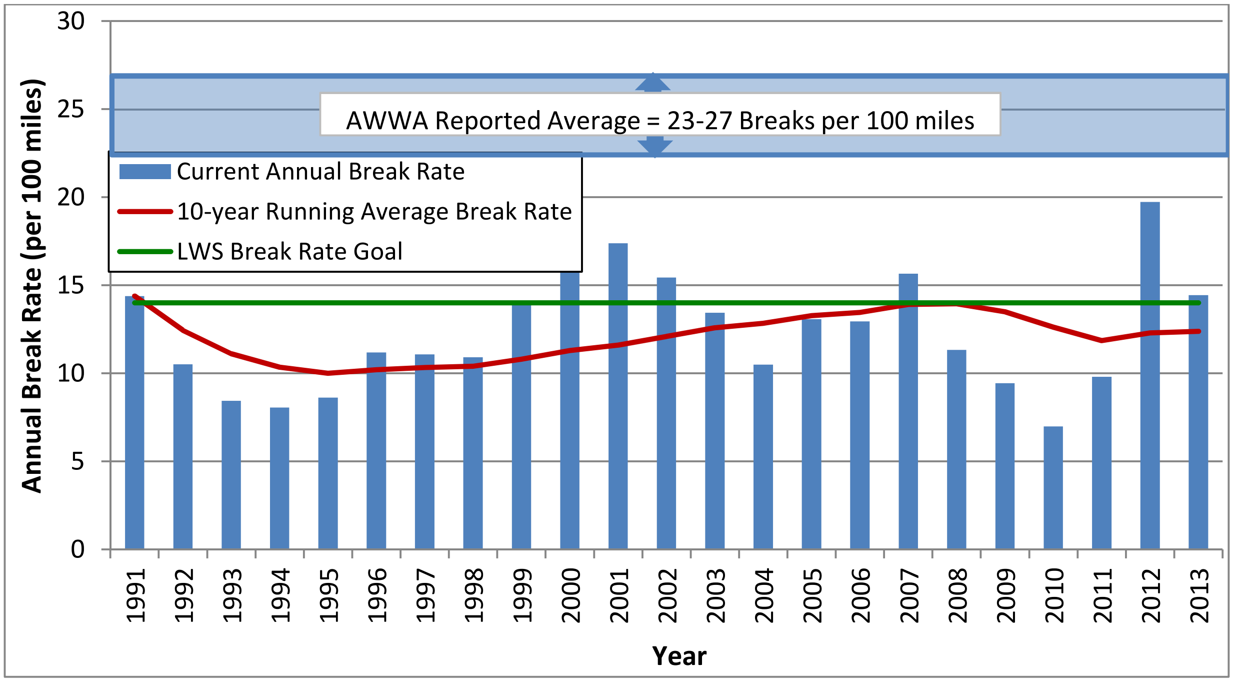

Figure ES-6 System Performance Relative to LWS Goal & AWWA Recommended Goal

Based on LWS's current performance, desired level of service, cost targets, backlog of pipe requiring replacement, and risk tolerance, an investment level of approximately 7 miles of pipe replacement per year is recommended. This would require approximately $6.3 million per year in 2013 dollars. This level of replacement would maintain the current level of backlog for water mains requiring replacement.

7.0 Asset Management

The City performed a formal asset management needs assessment with CH2M Hill in 2009. Through this effort, a comprehensive asset management program was identified as both a gap and a priority for continuing high performance operations and organizational sustainability at LWS.

The methodology used as a part of this Master Plan to conduct the asset management evaluation for LWS compared currently used business processes against industry best practices in the context of the Asset Management Framework, as depicted in Figure ES-7.

The framework items identified in the blue boxes in Figure ES-7 were the focus of this study:

- Asset knowledge is generally obtained and maintained in the computerized maintenance management system (CMMS).
- People and processes drive the asset management program.

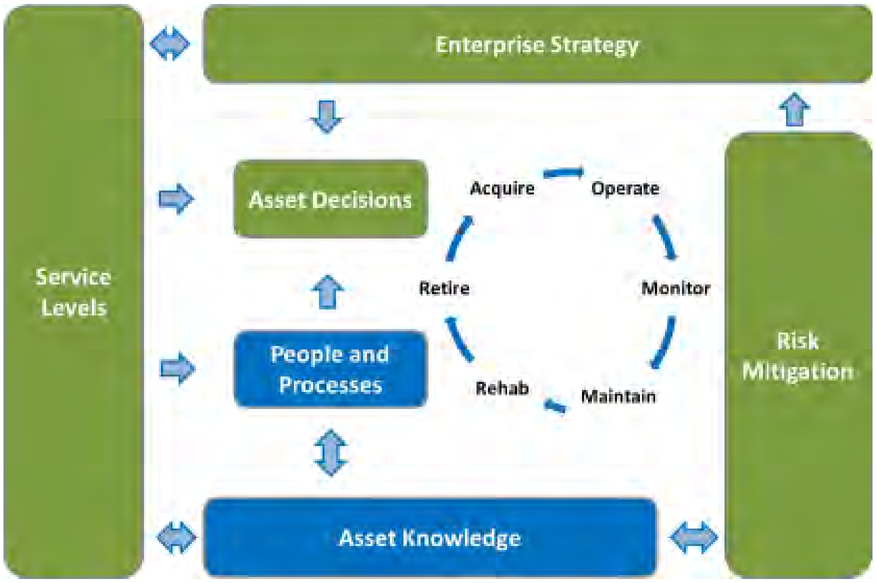

Figure ES-7 Asset Management Framework

The key tools currently used by LWS for asset knowledge include a CMMS program called Hansen and a geographic information system (GIS). LWS has made significant progress towards implementing these systems, but currently these systems operate independently of each other and are used inconsistently throughout the division.

LWS has an experienced staff, with much of the maintenance activities being conducted based on well developed schedules and system knowledge. While this knowledge and expertise is critical to the overall operation of the system, using the Asset Management Framework with the proper information technology (IT) systems and business processes will facilitate information transfer and will significantly reduce the potential of losing system knowledge as a result of staff changes.

A robust asset management program will provide LWS with the information and tools necessary to make critical decisions for the system. These decisions include maintenance scheduling and proactive prioritization of capital renewal and replacement projects.

LWS has made progress in the implementation of an asset management program with the further implementation and population of Hansen and GIS. To further advance this program, LWS should develop defined and consistent business processes throughout all sections within LWS. This includes consistent use of CMMS and GIS, establishment of an asset management hierarchy, and routine syncing of the CMMS and GIS. In addition, an asset management project leader should be identified to facilitate the implementation of the program.

Another critical element of a robust asset management program is the implementation of a condition assessment process. This process will allow LWS to further extend the useful life of assets, reduce the potential for failure, and identify those assets that have the highest potential for and consequence of failure in the system.

8.0 Financial Assessment

The objective of the financial assessment is to provide a conceptual review of the financial feasibility of the Master Plan. This financial assessment is not a comprehensive rate study, and it is not intended to be used for rate setting purposes. The financial assessment considers both the annual operating costs and capital needs of LWS. The financial assessment determines the financial feasibility of the Master Plan and what adjustments may be needed to the current water rates and revenues to adequately support the Master Plan. At the same time, the financial assessment considers other financial planning criteria, such as debt service coverage (DSC) covenants and maintenance of adequate reserve levels.

Two time periods were explored for the financial assessment: a 10-year projection and a 30-year projection. The 10-year projection is more critical for immediate financial planning purposes, but developing a financial forecast for a 30-year time period allows for a longer range look at the needed improvements and costs to the system. The financial feasibility of the Missouri River Project, which carries an additional and significant financial burden with it, was also explored.

8.1 10-Year Projection

Capital improvement projects during the first 10 years, from FY 2014 through FY 2023, total approximately $235 million. The capital projects in any particular year range from approximately $10 million to slightly over $40 million. Given that LWS is currently funding approximately $10 million per year for capital projects, the funding of the larger projects in the Master Plan will likely need to be debt funded. In the model developed as a part of the financial assessment, the overall annual debt service payments are projected to increase during this 10-year period from approximately $5.0 million per year to slightly over $9.0 million per year.

In addition to debt funding, the Master Plan capital projects during this period will be funded on a “pay-as-you-go” basis using rate revenues. This will require LWS to continually increase the level of funding of the CIP from rates over this 10-year period from the current level of approximately $10 million per year to about $15 million per year in FY 2023. The needed rate adjustments to support the Master Plan for FY 2014 through FY 2023 average 5 percent over this time period. Inflation alone accounts for approximately 3 percent per year in rate adjustments. During this time period, renewal and replacement projects for the existing LWS system account for over 50 percent of the CIP.

8.2 30-Year Projection with the Missouri River Project

During the 30-year time period, from FY 2014 through FY 2044, the CIP for LWS has approximately $1.0 billion in capital projects without the addition of a long-term water supply required to meet the needs of the system in the mid-2040s. The identified long-term supply project, the Missouri River Project, has a 2013 (estimated) cost of approximately $500 million, and when escalated to the end of the 30-year period, it approaches $1.2 billion. When placed in this context, the Missouri River Project essentially doubles LWS's capital plans over this 30-year time period.

A project of this magnitude raises a number of serious financial questions. Most importantly is whether this project is "affordable" and, if so, whether there is a financial strategy that LWS should consider for this particular project. There are no simple strategies to fund a project of this magnitude. However, it has concluded that LWS should consider the following strategy:

- As soon as possible, LWS should begin to set aside funds in a dedicated Missouri River Project reserve. The intent of this reserve is to begin to pre-fund the project such that it does not require 100 percent debt financing in 2040 or when built.
- The intent of funding this reserve is two-fold. First, it sets aside funds for the project, but more importantly, it begins to ramp up LWS's rates to a point at which LWS can support the eventual debt service payments associated with the project. Once the project is built, the financial strategy is that LWS will have gradually built into its rates, over the last 30 years, an amount that will pay a substantial portion of the annual debt service payment going forward. The key to this strategy is that it should minimize the need for a major rate adjustment (for example, a doubling of rates in a single year) at the time the debt is issued.
- A significant amount of funds will need to be collected annually and set aside in this dedicated reserve. Even with these funds set aside, LWS may be able to fund only 10 to 15 percent of the total expected project costs from this reserve.
- When the Missouri River Project is being built, LWS should deplete the dedicated reserve and apply those reserves against the project. The balance of any needed funds to construct the project will be obtained from the issuance of long-term debt.

While this strategy appears to be relatively sound on the surface, it will likely be more complicated in reality. In particular, asking today's customers to fund a project that is potentially 30 years into the future, and may or may not be built, creates a certain set of political challenges on its own. Though not impossible, it may be difficult for LWS to start the reserve in the near future; instead, LWS may need to wait until there is greater certainty around the Missouri River Project. However, that strategy has its own pitfalls in that the amount of funds collected in the dedicated project reserve may be minimal due to the shortened amount of time available to accumulate funds. Alternatively, the size of the rate adjustments needed over the shorter time period to ramp up to the anticipated level of debt service may be too large on an annual basis. The financial assessment developed in this Master Plan is intended only to answer the basic question of whether it is potentially feasible to be fund the Missouri River Project.

The scenario considered assumed that LWS would begin to fund the Missouri River Project reserve in FY 2018 at $1 million per year. Over time, the annual contribution would increase to $32 million by FY 2040. At that point, the Missouri River Project reserve would fund approximately $298 million of the project costs. Additional funds would be collected from rates during the construction period, and an additional $25 million is assumed to also be available in construction reserves. When taken together, this is approximately 28 percent of the anticipated project cost, meaning that the balance of approximately $882 million (72 percent) would need to be funded from long-term debt.

From the projection of revenues and expenses, along with a funding plan for CIP projects, a summary of the revenue requirements for LWS for the 30-year period from FY 2014 through FY 2044 was developed. This summary is provided in Figure ES-8.

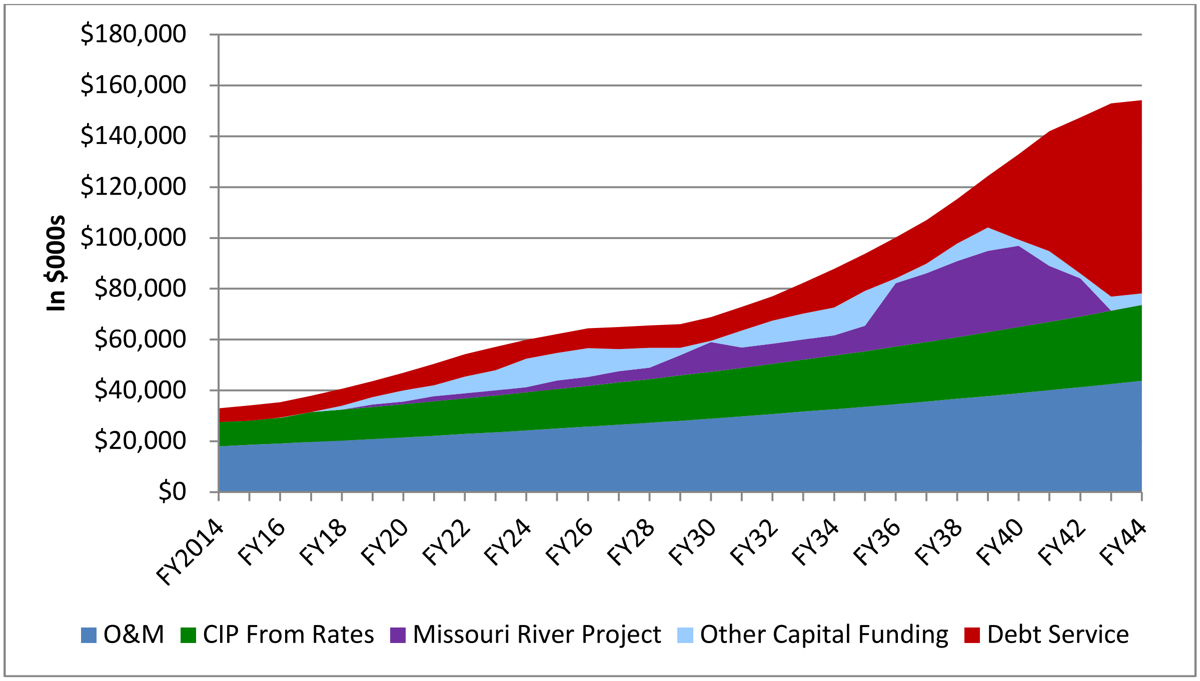

Figure ES-8 Summary of the 30-Year Financial Assessment ($000) Including the Missouri River Project

As shown in Figure ES-8, the purple area is the funding of the Missouri River Project reserve. In addition, the light blue funding of the “other capital funding” helps to position LWS for the eventual debt service that occurs, as shown in dark red. By FY 2044 and using the assumptions discussed above, the total revenue requirement is approximately $160 million.

While the magnitude of the dollars is exceptionally large in relation to today's costs, the needed rate adjustments, on their own, appear feasible and manageable. The financial assessment assumes needed rate adjustments on average of 4.5 to 5 percent over this time period.

While the level of the rate adjustments appears to be reasonable, the potential impact on a typical residential customer's bill was reviewed, as shown in Figure ES-9. This figure shows both nominal and real dollars in the form of net present value (NPV).

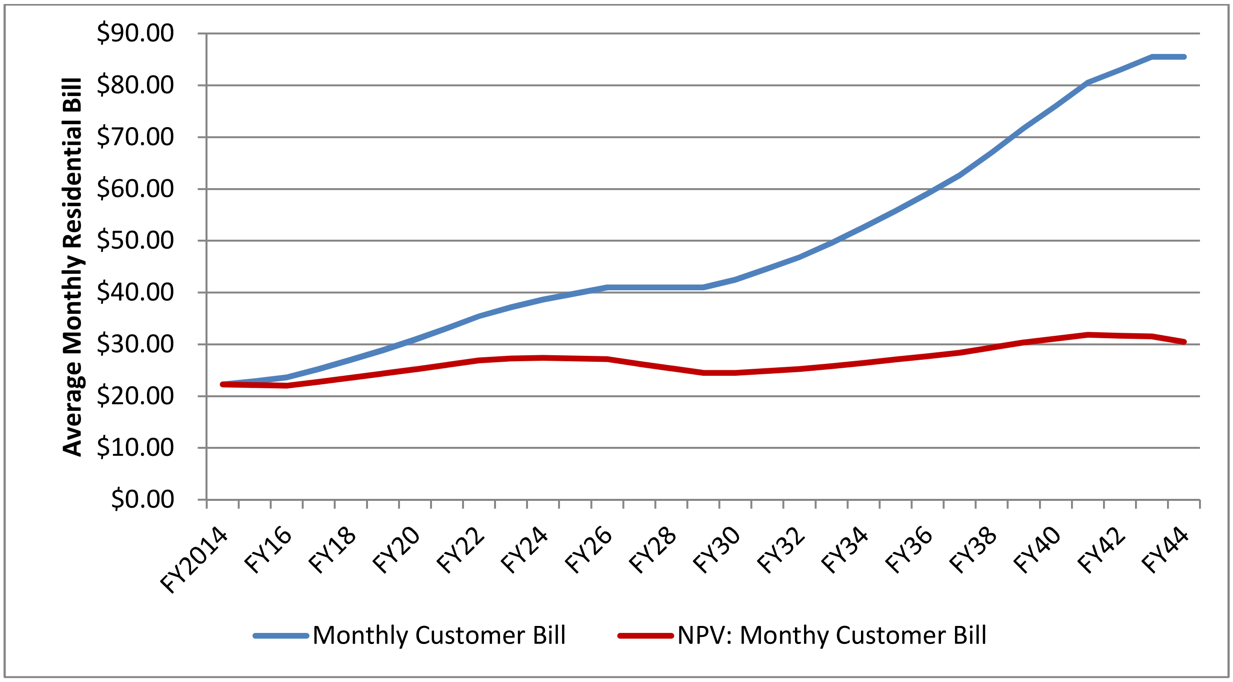

Figure ES-9 Projected Average Residential Monthly Bill – FY 2014 – FY 2044 Including the Missouri River Project

At the present time, the average monthly residential bill is approximately $22.24. Assuming the annual rate adjustments shown in Figure ES-9, the average monthly residential bill could increase to approximately $85.49 per month. If this value is adjusted (deflated) for the assumed time value of money, then, in net present value (NPV) dollars, the cost is approximately $30.46 per month. The assumed discount rate used for the present value analysis was 3.5 percent. This result is subject to the variability of the assumptions used over the 30-year period, and the result may vary significantly under different assumptions.

8.3 Affordability Issues

Affordability is a concern of all utilities given the fact that rates and charges for utility services have been increasing at a pace that exceeds the cost of living (CPI). Affordability has now come to the forefront of many discussions, particularly as it relates to major capital infrastructure funding and financing. Affordability for the community is defined as a percentage of the median household income (MHI). Average residential bills which exceed this threshold are considered “unaffordable”. Typical measures used have ranged from 1.5% to 2.5% of a community’s MHI.

In the case of the City, the median household income is approximately $46,600. Using a 2.0 percent measure, this means that the average residential bill would need to be $77.00 per month before the rate would be considered “unaffordable” on a community-wide basis. Stated another way, the current average residential bill of $22.24 is approximately 0.5 percent of the average MHI for Lincoln, which is in the low financial impact range. Given LWS’s currently low rates, it would seem that nothing within this financial assessment that would indicate that the Master Plan is unaffordable on a community-wide basis. However, at an individual level, there may be affordability issues. As LWS’s rates continue to increase over time, LWS and the City may consider different methods for addressing the needs of these specific customers.

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Lincoln Water System Facilities Master Plan

Chapter 1 - Introduction

May 2014

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Abbreviations and Acronyms

CIP	Capital Improvements Program
City	City of Lincoln
LWS	Lincoln Water System
Master Plan	2013 Facilities Master Plan

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1.0 Introduction

Water utilities must continuously plan to address system needs and challenges, such as system growth, aging infrastructure, increasingly stringent regulatory requirements, and the need for a well-planned and efficient capital improvements program (CIP). Recognizing this need, the City of Lincoln (City) has historically conducted master planning efforts at 5-year intervals: a comprehensive master planning effort every 10 years and updates to address system growth and distribution system needs every 5 years. The City last completed a comprehensive Facilities Master Plan in 2002 and an update in 2007.

The City retained HDR Engineering, Inc. to prepare the comprehensive 2013 Facilities Master Plan (Master Plan). This Master Plan will provide a guide to the short-term and long-term improvements for the infrastructure of the Lincoln Water System (LWS) through the year 2060.

2.0 Scope

The major components of the Master Plan include the consideration and evaluation of the following:

- Water Capacity Requirements – use historical water usage trends to develop future projections for water usage and demand based on future population projections.
- Water Supply – evaluate the adequacy of the existing supply to meet current and future water supply needs. Evaluate alternatives to enhance the long-term reliability and sustainability of the City's water supply.
- Water Treatment – assess treatment needs to address existing and future regulatory and capacity requirements.
- Transmission and Distribution Systems – summarize existing transmission and distribution systems infrastructure and operations and analyze the capacity of the systems to meet present and future demands.
- Water Main Replacement Program – assess the existing water main replacement program, review main break history and distribution system maintenance and determine the level of investment required to sustain the City's water main infrastructure.
- Asset Management Program – develop and summarize a framework for development and/or enhancement of the City's water system condition assessment and asset management programs.
- Financial Assessment –review the revenues and expenses for LWS. The capital costs contained within the financial assessment are based on the Capital Improvement Program (CIP) developed within the Master Plan.

At the completion of the evaluation and analysis of each of the key master plan components described above, a Chapter of the Master Plan was prepared and submitted to the City for review. After the review process the final chapters were compiled to form the final Master Plan.

3.0 Study Area

The study area for the Master Plan is time dependent. As the planning horizon moves out in time the study area grows to cover the anticipated growth in the City. The planning horizons used in the Master Plan have been coordinated with the Lincoln/Lancaster County 2040 Comprehensive Plan (LPlan 2040) as follows:

- Tier I Priority A – This includes areas currently under development.
- Tier I Priority B – This includes areas expected to be developed by 2025.
- Tier I Priority C – This includes areas expected to be developed by 2040.
- Tier II – This includes areas expected to be developed by 2060.

The corresponding area is presented in Figure 1-1.

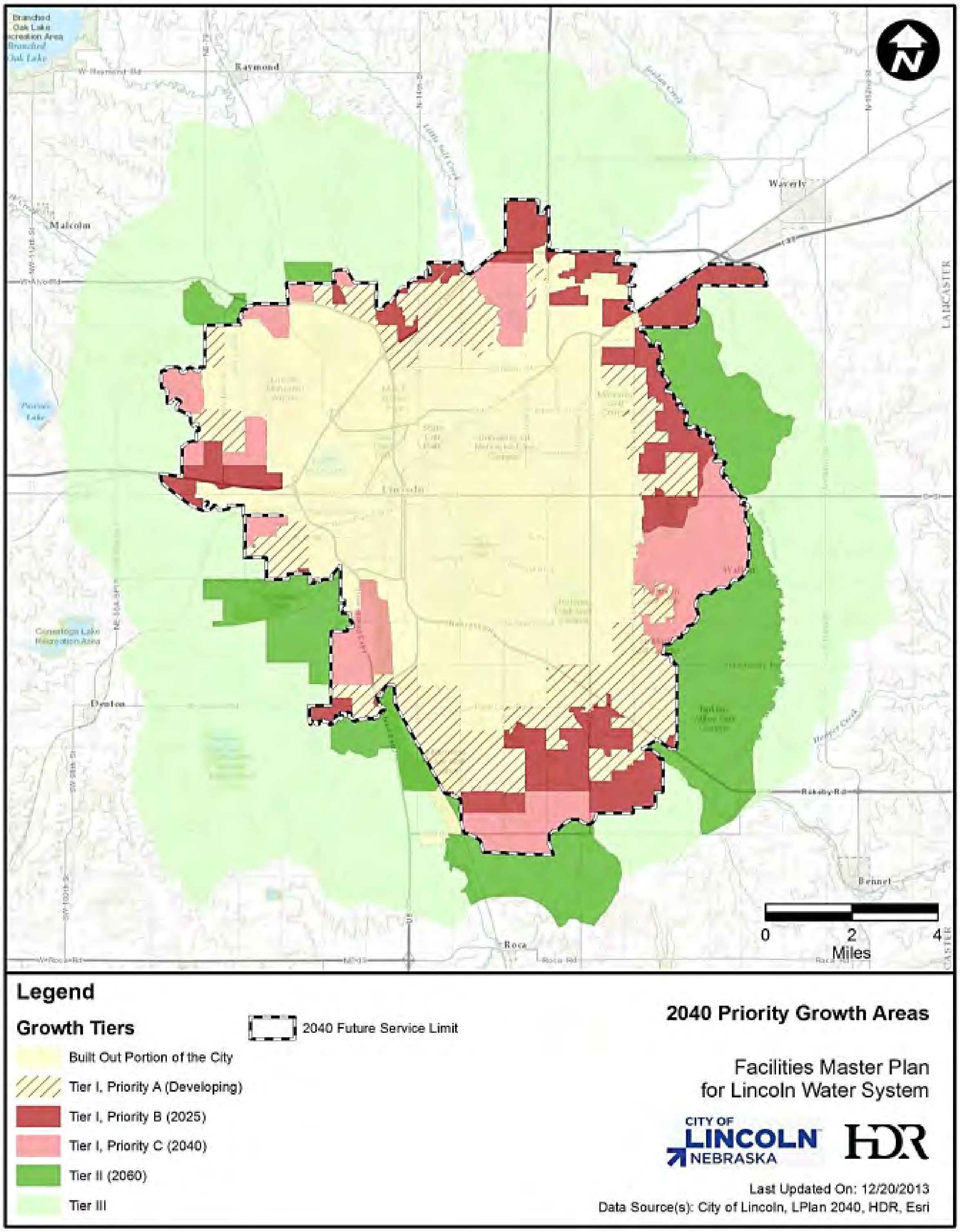

Figure 1-1 Study Area

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Lincoln Water System Facilities Master Plan

Chapter 2 - Water Capacity Requirements

May 2014

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Abbreviations and Acronyms

ADD	Average Day Demand – The total annual demand divided by 365 days (or 366, if a leap year).
CIP	Capital Improvements Program
City	City of Lincoln
FY	Fiscal Year – September 1st to August 31st
gal	Gallon
gpcd	Gallons per Capita per Day
GIS	Geographic Information System
2002 Master Plan	2002 Facilities Master Plan
2007 Master Plan	2007 Facilities Master Plan
Master Plan	2013 Facilities Master Plan
LPlan 2040	Lincoln/Lancaster County 2040 Comprehensive Plan
LWS	Lincoln Water System
MDD	Maximum Day Demand – The total daily demand during the day with the greatest amount of demand in a given year.
MG	Million Gallons
mgd	Million Gallons per Day
MHD	Maximum Hour Demand – The water demand during the hour with the highest system demands in a given year.
NRW	Non-Revenue Water – Water used for purposes that is not billable i.e. fire fighting, flushing, main breaks and leaks, etc.
SCADA	Supervisory Control and Data Acquisition
SP	Seasonal Peak – The average use over the highest consecutive three months of raw water supply.
TAZ	Transportation Analysis Zones
WTD	Well Field, Transmission and Distribution
WTP	Platte River Water Treatment Facility
YR	Year

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1.0 Introduction

The purpose of *Chapter 2 – Water Capacity Requirements* is to establish the future system capacity requirements for the Lincoln Water System (LWS) through the year 2060. This planning effort includes an evaluation of the population densities and potential growth areas within the City's existing and proposed future service areas. Based on these population projections and historical water usage trends, demand projections were developed. These projections will serve as the basis for evaluation and planning in the subsequent Chapters of the 2013 Facilities Master Plan.

2.0 Population

Population projections and growth patterns are used as the basis for developing plans for the LWS to serve growth and for analyzing impacts to the existing conveyance, treatment and supply infrastructure. The Lincoln/Lancaster County 2040 Comprehensive Plan (LPlan 2040), as adopted in October 2011, includes projected populations through the year 2040 and includes anticipated growth areas through the year 2060. LPlan 2040 will serve as the basis of analysis of the projected populations and service area needs for the LWS.

2.1 Planning Period

The planning period for this master planning effort is from the year 2012 through the year 2060. This planning period will be used for evaluating improvements or service expansions. Year 2012 will serve as the base year for this analysis. Demand projections and distribution analysis will be conducted for three specific planning intervals:

- 2025 (short-term)
- 2040 (mid-term)
- 2060 (long-term)

These planning periods were selected in coordination with the LPlan 2040.

2.1.1 2025 (Short-Term)

The short-term analyses provide recommendations for both improvements to address existing system deficiencies and expanding facilities to serve short-term new development areas. For this timeframe, recommended improvements are prioritized, and construction phasing and timelines are developed. Recommended improvements are summarized in a Capital Improvements Program (CIP) along with estimated capital costs.

2.1.2 2040 (Mid-Term)

The mid-term analyses provide an interim benchmark between short-term facility improvements and long-term goals. These analyses provide a basis for the timing of phased improvements and provide a measure of how soon major improvements may be required after the short-term period. Recommended improvements are prioritized and capital improvement cost estimates are provided for general planning purposes.

2.1.3 2060 (Long-Term)

The long-term analyses are primarily provided as a basis for evaluating how long-term growth may impact LWS facilities. Population projections and future development cannot be accurately quantified for this 50-year horizon, but the projections will help identify potential shortfalls in the LWS. The long-term analyses provide a basis for evaluating long-term requirements for raw water supply, treatment plant, and water transmission. The long-term plan provides a foundation for phasing of improvements and helps avoid installing short- and mid-term improvements that may not account for long-term needs. Detailed CIPs will not be developed for the long-term period.

2.2 Study Area

The study area for the Master Plan includes the area encompassed by the city limits for the City of Lincoln (City), both existing and future. LPlan 2040 delineates the current City limits along with the anticipated future service limits through the year 2040. In addition, two additional tiers of growth areas beyond 2040 were delineated. As presented on Figure 2-1, these limits and growth areas include:

- **Existing City.** Built out portions of the City as of 2011.
- **Future Service Limit through the year 2040 as defined by the Tier I growth area.** This growth area was further divided into development priority areas:
    - **Tier I – Priority A (current development):** Comprised of undeveloped land within the existing City limits, as well as areas that are not yet annexed but which have approved preliminary plans. The Tier I – Priority A area includes approximately 22.5 square miles.
    - **Tier I – Priority B (2025):** The next priority of development which is generally contiguous to existing development and should include installation of the required water system infrastructure by the year 2025. The Tier I – Priority B area encompasses approximately 17.7 square miles.
    - **Tier I – Priority C (2040):** The last priority of the Tier I growth area that includes those areas which currently lack almost all infrastructure required to support urbanization. These areas are anticipated to develop towards the end of the Tier I growth period, beyond 2025, with the required water system infrastructure being installed by the year 2040. The Tier I – Priority C area includes approximately 16.5 square miles.
- **Tier II, 50-year Long-Term Potential Service Area**. **Tier II** is the next area of development and is anticipated to occur beyond the 30-year planning horizon of LPlan

2040 to 50 years, and possibly further. This area includes approximately 34 square miles. For the purpose of the Master Plan, this area will be considered for long-term utility planning for supply development, water treatment needs and major infrastructure improvements through the year 2060.

- **Beyond 50-year Service Area.** LPlan 2040 defines a **Tier III** growth area which is beyond the planning horizon and is not considered in the Master Plan.

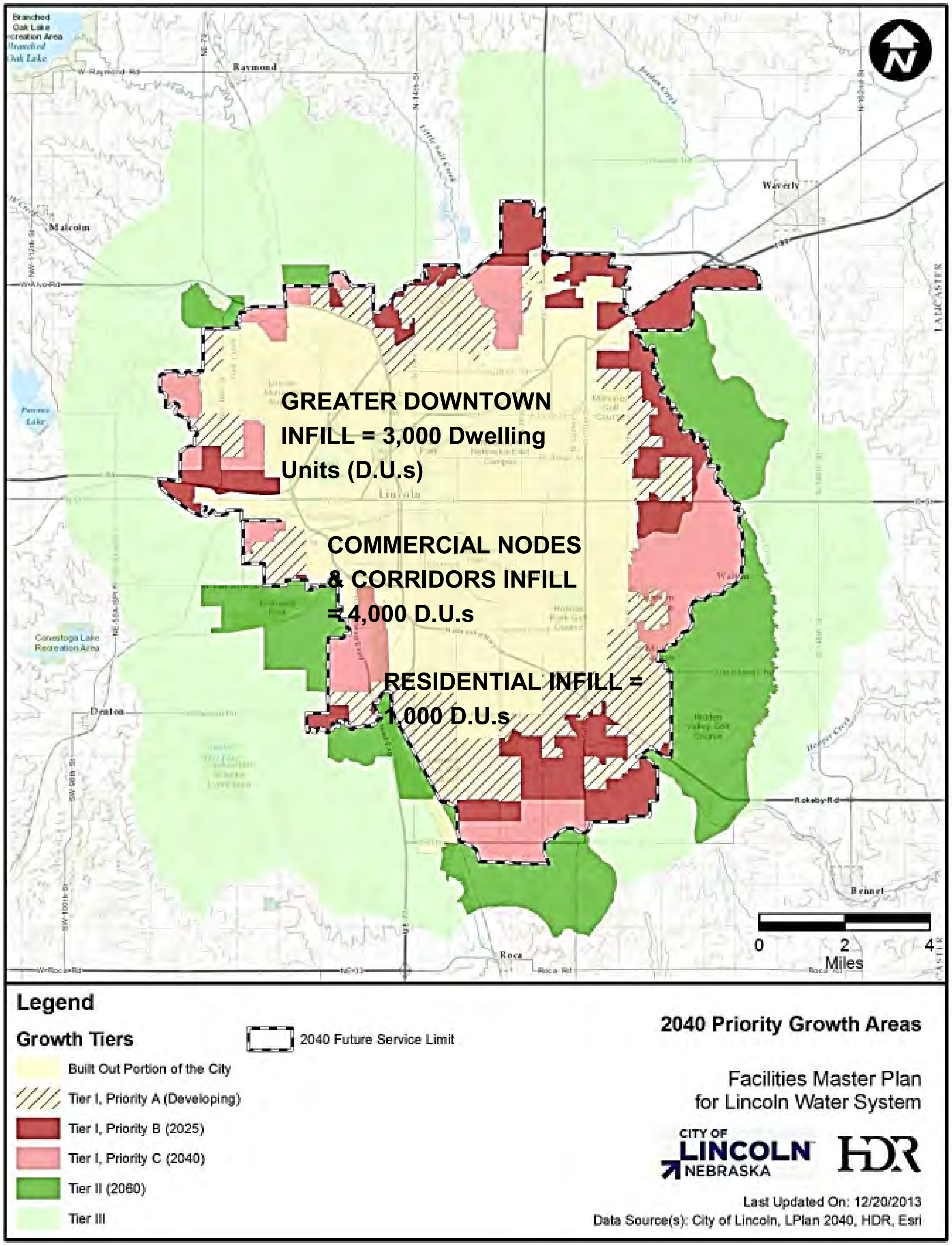


Figure 2-1 2040 Priority Growth Areas

2.3 Historic and Future Population

Census data from the past 70 years was used to establish the population history for the City. In 1940, the population was 81,984 people. In 2010, the population had grown by 176,395 people to reach 258,379. This represents an average annual growth over the past 70 years of 1.65%. For the past 40 years, the population density has been approximately 3,000 people per square mile.

The LPlan 2040 projected the population for the City to increase by 1.2% annually for planning years 2040 and 2060, which is equal to 371,700 people in 2040 and 461,700 people in 2060. Similar to the LPlan 2040 projections, the 1.2% annual growth rate was used to determine the population for the base year 2012, the Tier I – Priority B (2025) and populations for years 2020, 2030, 2050. The historic and projected populations are summarized in Table 2-1 and presented graphically in Figure 2-2.

Table 2-1 Historic and Projected City of Lincoln Population

Year	Population	Average Annual Growth	
		Persons	%
1940	81,984[1]	--	
1950	98,884[1]	1,690	1.9
1960	128,521[1]	2,964	2.7
1970	149,518[1]	2,100	1.5
1980	171,932[1]	2,241	1.4
1990	191,972[1]	2,004	1.1
2000	225,581[1]	3,361	1.6
2010	258,379[3]	3,280	1.4
2012 (base year)	264,618[2]	3,119	1.2
2020	291,100[2]	3,272	1.2
2025 (short-term)	309,000[2]	3,580	1.2
2030	328,000[2]	3,800	1.2
2040 (mid-term)	371,700[4]	4,370	1.2
2050	416,400[2]	4,470	1.2
2060 (long-term)	461,700[4]	4,530	1.2

Notes:

1. *Obtained from Lincoln Water System 2007 Facilities Master Plan.*
2. *Population interpolation based on annual growth rate of 1.2 percent*
3. *Based on data for U.S. Census Bureau*
4. *Projections included in LPlan 2040, based on annual growth rate of 1.2 percent..*

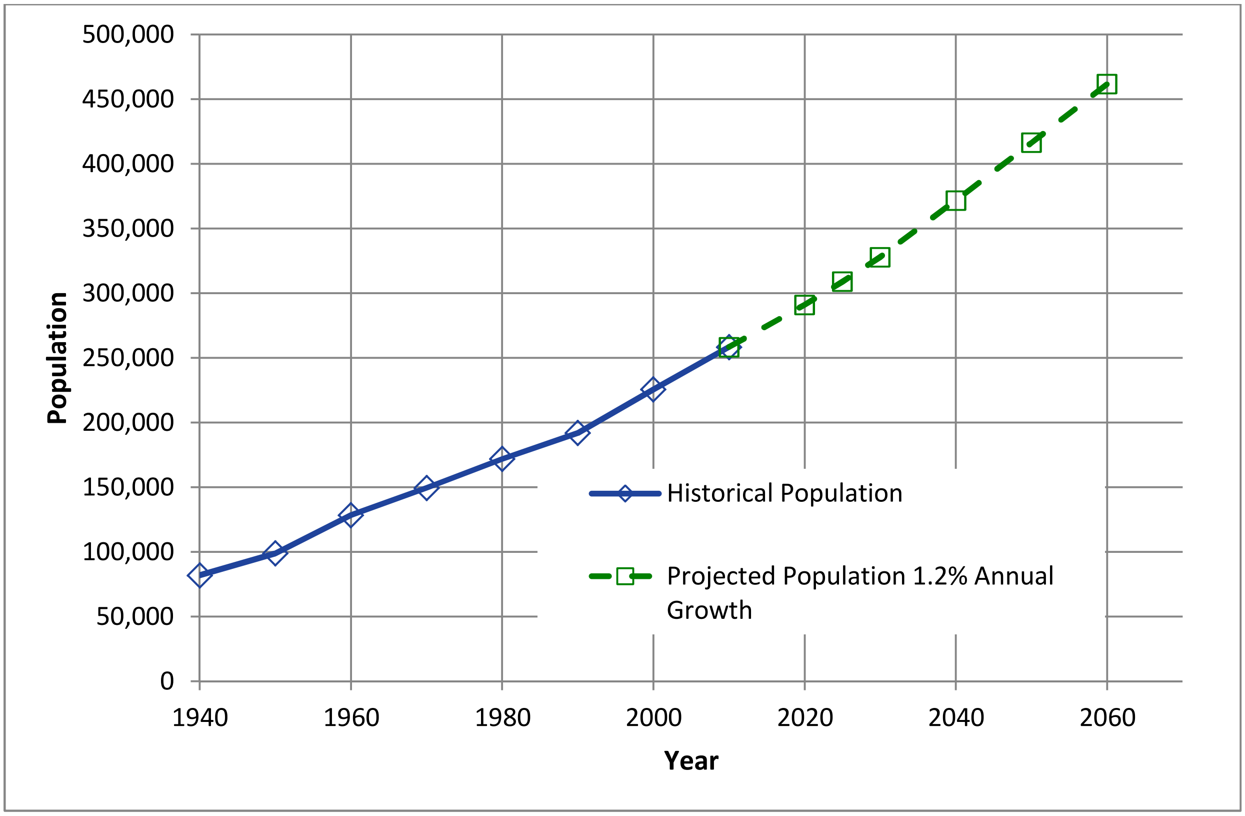


Figure 2-2 Historic and Projected City of Lincoln Population

2.4 Population Distribution

2.4.1 Distribution Analysis

Lancaster County data for number of households in 2010 and population projections for years 2025 and 2040 was provided by the Lincoln-Lancaster County Planning Department. This data was distributed spatially over the system using a Geographic Information System (GIS) with the provided transportation analysis zones (TAZ). The 2010 households and 2025 and 2040 population projections were delineated by the Lincoln-Lancaster County Planning Department into total of 502 TAZ areas that covered the entire planning area, including the Tier III growth area limits. However, since this study only includes up to the Tier II boundary, the TAZ areas were adjusted accordingly, leaving a total of 431 TAZ areas for the planning horizon of the Master Plan.

The population for each TAZ was calculated for years 2010, 2012, 2025 and 2040. The 2010 population was derived from the number of households in each TAZ, the percentage of single-family households and multi-family households in each TAZ and an estimate of 2.61 people per single-family household and 1.77 people per multi-family household as provided by the Lincoln-Lancaster County Planning Department. The estimates of people per single-family household and people per multi-family household were then adjusted to 2.58 and 1.74, respectively, to match existing 2010 populations within the existing service area. The 2012 population in each TAZ was established by adjusting the population to match an overall system-wide growth rate of 1.2% from 2010 to 2012.

The 2025 and 2040 population for each TAZ was already calculated in the original TAZ data provided by the Lincoln-Lancaster County Planning Department. The 2025 and 2040 TAZ populations account for system expansion, as well as in-fill and redevelopment in previously developed areas.

TAZ data was not available beyond year 2040, so the population for each TAZ for year 2060 was forecasted by allocating the aggregate population increase between 2040 and 2060; half of the population increase was assigned to TAZs that showed growth between 2025 and 2040 to capture in-fill and redevelopment potential and the other half of the population increase was assigned to the Tier II (2060) boundaries based on area to capture expansion areas.

2.4.2 Population by Service Level

The historic population, by service level, was calculated for the years 2010 and 2012 for this Master Plan and is based on the population counts by TAZ. The percentage of population served was based on existing service levels which encompassed the existing City and Growth Tier I boundaries. From the resulting populations by service level, slight adjustments were made to match the system-wide 2010 and 2012 population based on the service level annual growth rates over the period from 2006 to 2010. The historical population by service level from 1980 to 2012 was tabulated as shown in Table 2-2. The Cheney and Northwest service levels are relatively new growth areas for the City and were created after the 2000 census.

Table 2-2 Historic Population by Service Level

Service Level	1980[1]	1990[1]	2000[1]	2006[1]	2010[2]	2012[3]
Northwest	-	-	-	1,765	2,173	2,299
Belmont	14,500	18,890	31,830	34,609	38,693	40,922
Low	64,800	67,100	71,466	76,668	76,729	76,760
High	81,600	89,210	94,840	100,908	101,251	102,265
Southeast	12,350	16,770	27,455	30,377	36,209	38,382
Cheney	-	-	-	2,372	3,324	3,990
Total	**173,250**	**191,970**	**225,581**	**246,699**	**258,379**	**264,618**

Notes:

1. *From 2007 Facilities Master Plan.*
2. *Calculated based on number of single-family and multi-family households and people per household in 2010 by TAZ in each service level.*
3. *Calculated based on 2010 Population by Service Level and an overall system-wide 1.2% growth rate.*

For years 2025 and 2040, the TAZ populations from LPlan 2040 adjusted to the existing City and Growth Tier I boundaries were used as the projected population served. The population by service level was calculated similar to years 2010 and 2012, except that the population served was adjusted to match the projected year 2025 and 2040 City population of 309,000 and 371,700, respectively. Figure 2-3 presents the service levels with existing and Tier I growth.

For year 2060, the established TAZ populations, as described previously, adjusted to the existing City and Growth Tiers I and II boundaries were used as the projected population served. The service levels were expanded out to the Growth Tier II boundary based on surrounding topology and service level expansions from previous planning efforts. The population by service level was then calculated based on the expanded service levels with slight adjustments made to match the aggregate population projection of 461,700 for 2060. Figure 2-4 presents the service levels with existing and Tier II growth.

Table 2-3 presents a summary of projected populations for each service level for each of the planning years for this study.

Table 2-3 Existing and Projected Population by Service Level

Service Level	Base Year (2012)	Short-term (2025)	Mid-term (2040)	Long-term (2060)
Northwest	2,299	4,900	6,500	9,400
Belmont	40,922	52,300	64,400	82,800
Low	76,760	77,300	89,800	100,800
High	102,265	113,100	126,300	147,700
Southeast	38,382	47,800	66,400	96,000
Cheney	3,990	13,600	18,300	25,000
Total	**264,618**	**309,000**	**371,700**	**461,700**

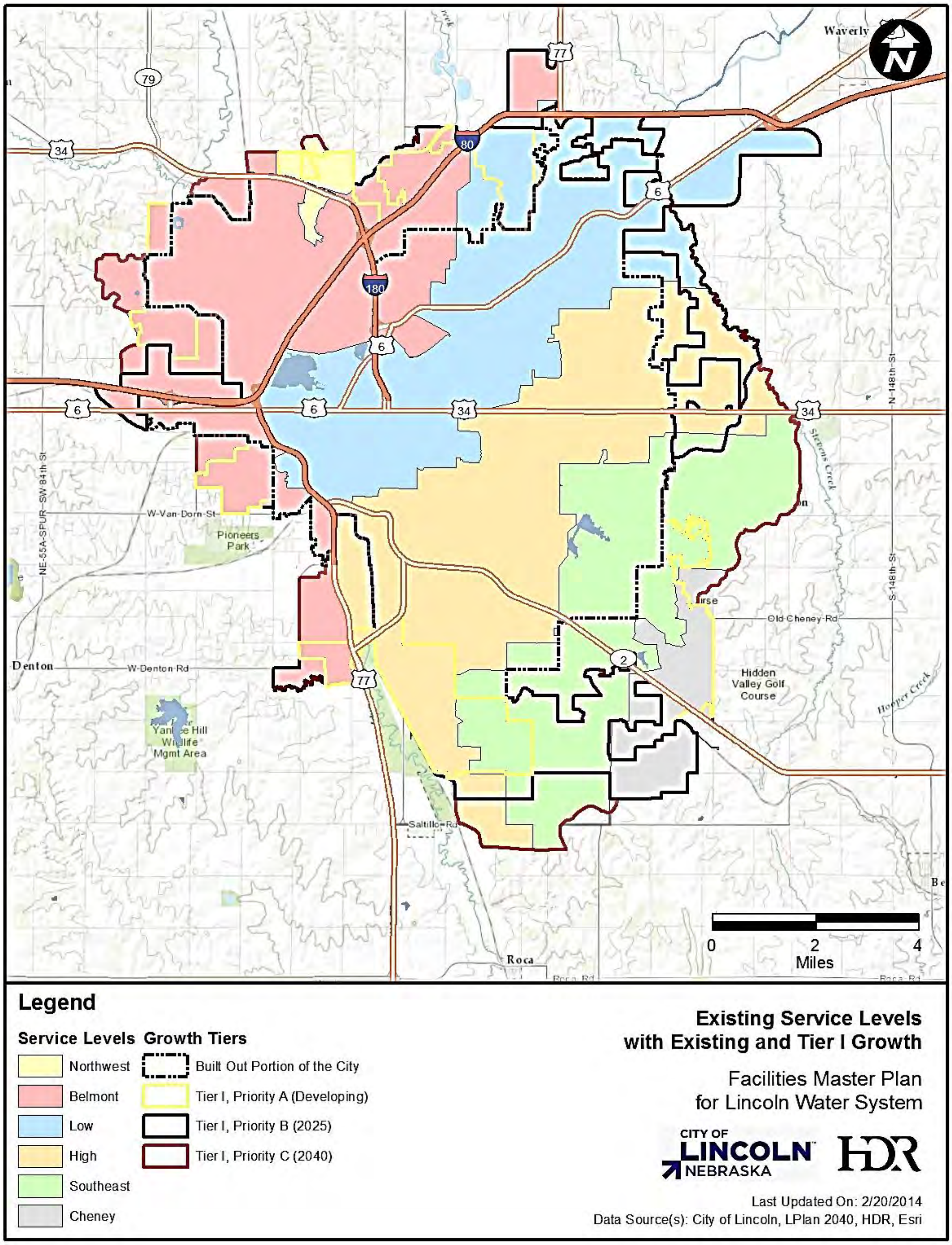


Figure 2-3 Service Levels with Existing and Tier I Growth

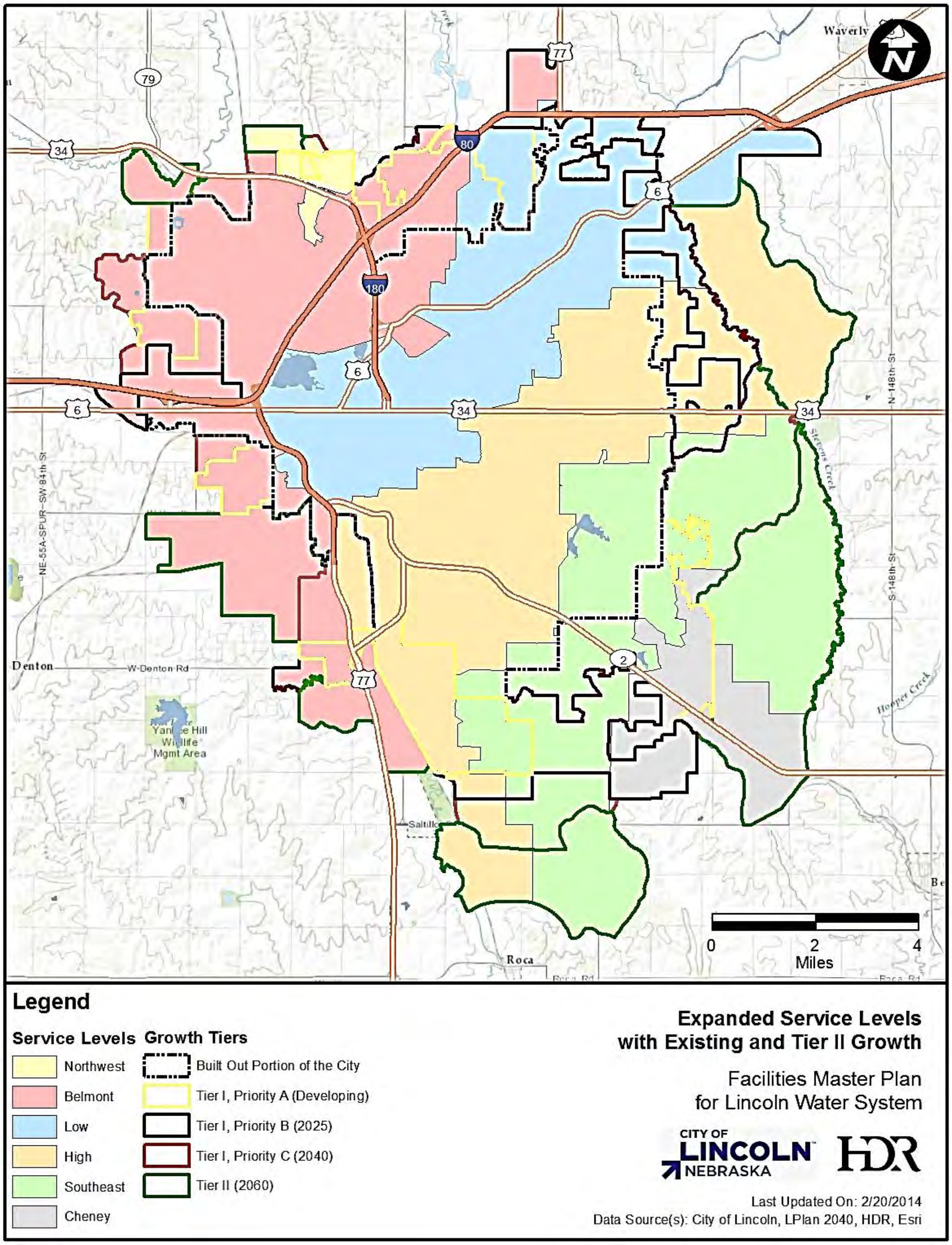


Figure 2-4 Service Levels with Existing and Tier II Growth

3.0 Water Capacity Requirements

Water capacity requirements for the planning period will be based on water demand and usage projections. These projections are developed through an evaluation of the historic water use trends and the population projections discussed in the previous section.

3.1 Methodology

Water demand varies throughout the year and can vary throughout any given day. There are diurnal, as well as seasonal, variations. During the summer months, water use typically increases relative to winter months. Water use also varies from year to year, depending on factors such as precipitation and temperature. In the assessment of the water system needs, the key water usage rates that need to be considered include:

- **Average Day Demand (ADD).** The ADD is the total water used during the year divided by 365 days per year. The ADD is used primarily to determine the adequacy of the water system to deliver the total amount of water needed during the year. It is also used as the common basis for developing peak demand projections. The ADD is the basis for estimating the maximum day and maximum hour demands and will be used for financial assessments for the system.
- **Maximum Day Demand (MDD).** The MDD is the maximum recorded daily demand, representing a single highest system demand for a given year. The water supply and treatment plant must be capable of supplying, treating, and transmitting enough water to meet the MDD.
- **Maximum Hour Demand (MHD).** The MHD is the water demand during the hour with the highest system demands. The distribution system must be capable of conveying water to customers at the MHD and at the LWS minimum design pressure of 45 psi. The MHD will be used to evaluate distribution system pressure, velocities and head loss in addition to storage equalization needs.
- **Seasonal Peak (SP).** The SP is the average daily use of raw water supply over the highest consecutive three month period. This period is typically June, July, and August or July, August, and September and SP is calculated by dividing the total raw water demand for this period by the number of days in the period (92 days). This value is used to assess the supply needs of the system for a prolonged period of time.

3.2 Historic Period of Record

When evaluating the historical water usage trends, it is important to evaluate water usage that is representative of conditions reflective of current usage and inclusive of variations in precipitation and temperature. As with many utilities, the LWS has seen a continued decline in per capita water usage based on improved water conservation throughout the community. To illustrate this, in 1990 the 5-year average per capita water usage was 174 gallons per capita per day (gpcd), in 2012 the 5-year average per capita water usage declined by 45 gpcd (25.9%) to 129 gpcd. As a result, many communities are electing to use a 10-year historical period of record for evaluation of water demand/usage patterns.

For the purpose of this Master Plan, the historical period of record for the water demand/usage evaluations was expanded to included fiscal year 2000 through fiscal year 2012. It was determined that this period of record would be reflective of the decline in per capita water demands, while also providing a generous cross-section of water use during wet years and dry years with higher temperatures. Figure 3-1 through Figure 3-4 present the annual precipitation and monthly precipitation and average high temperatures for July, August and September. As presented on these figures, this period includes several dry years with higher average temperatures during the summer months. The year 2012 was also one of the driest and hottest summers on record and as a result, water restrictions were implemented from July 11, 2012 through September 14, 2012. The most recent implementation of water restrictions prior to 2012 occurred in 2002.

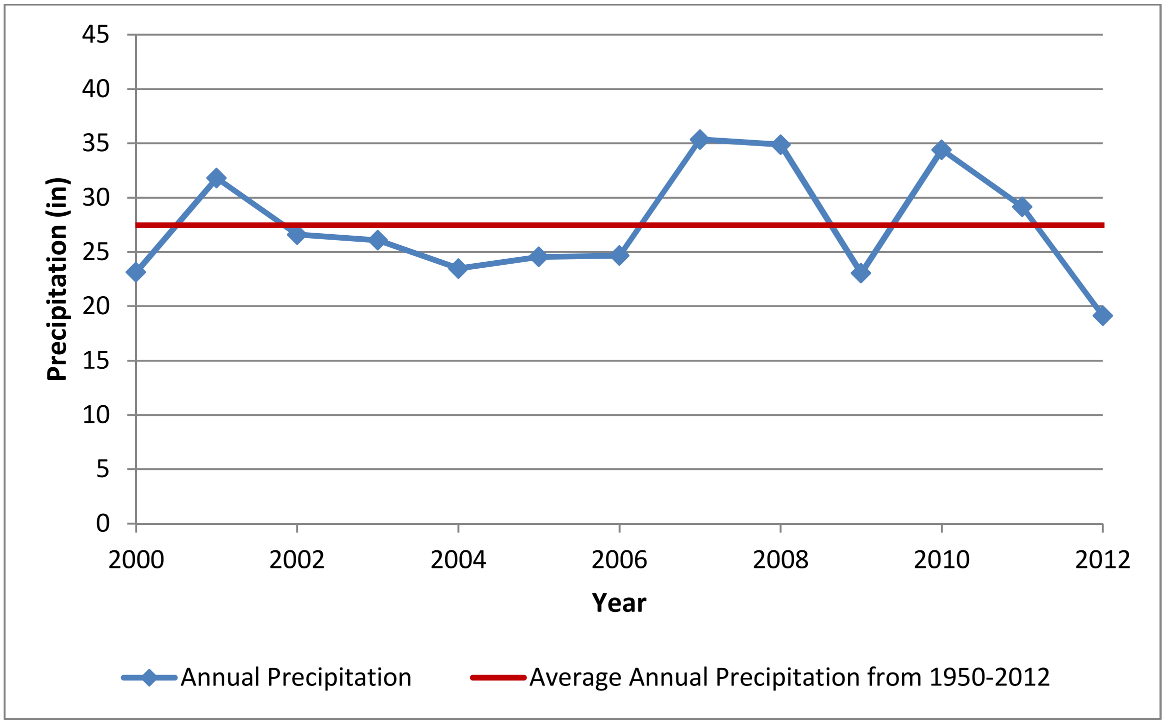


Figure 3-1 Lincoln Annual Precipitation

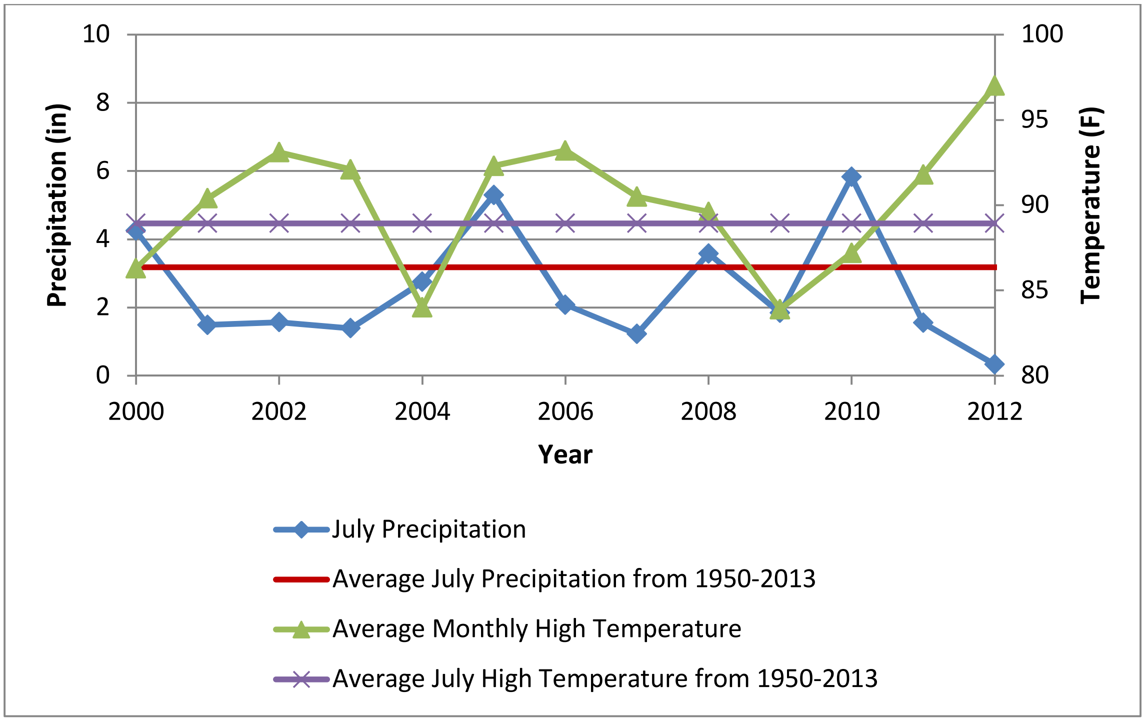


Figure 3-2 Lincoln July Precipitation & Average High Temperature

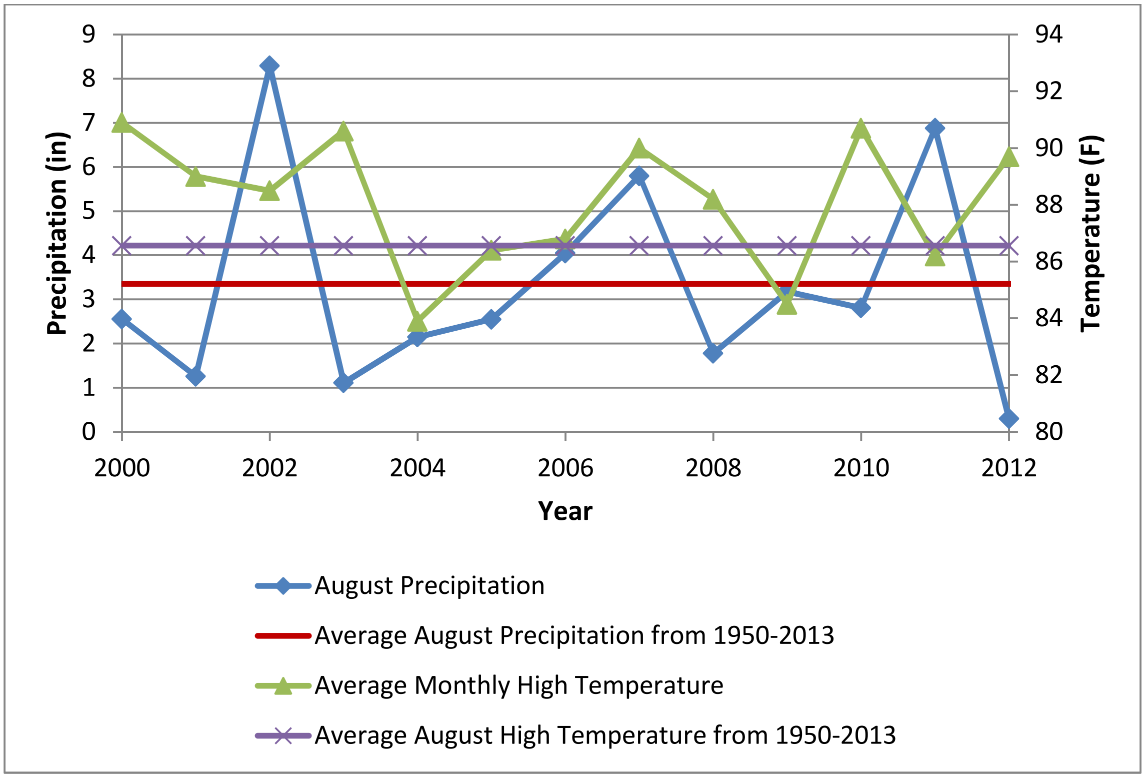


Figure 3-3 Lincoln August Precipitation & Average High Temperature

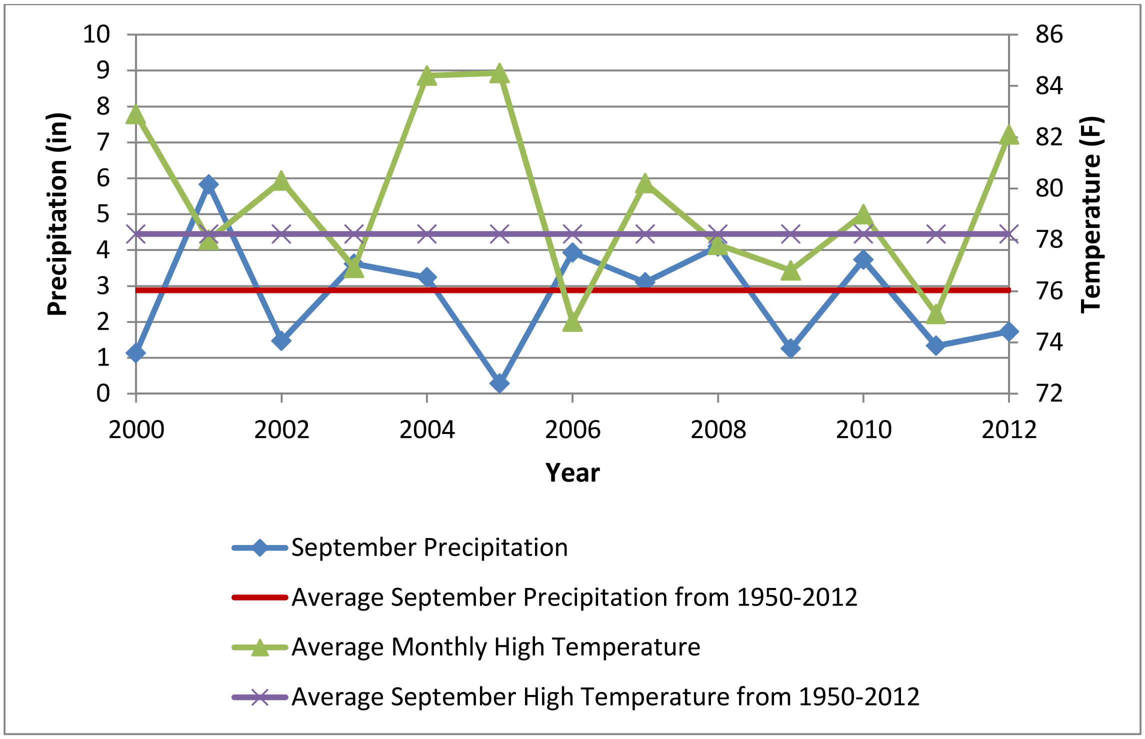
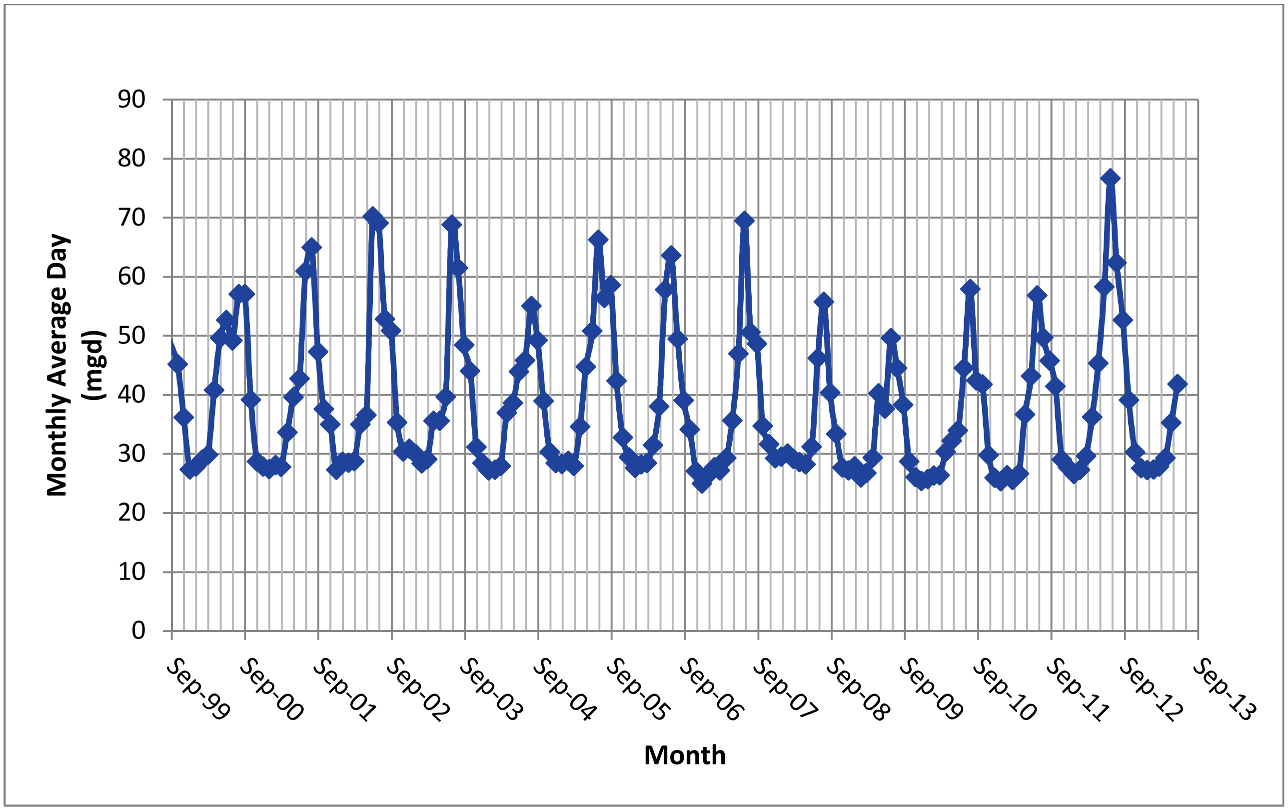


Figure 3-4 Lincoln September Precipitation & Average High Temperature

For comparison purposes and sensitivity analysis, the historical water usage trends were also evaluated for a 20-year historical period of record (1993-2012). The results of this sensitivity analysis are discussed in the following sections. However, for the basis of design for the Master Plan, the 2000-2012 historical period of record was used.

3.3 Historic Well Field Pumpage

To estimate future well production requirements, it is necessary to examine the historic well pumpage. To facilitate this, the City provided copies of the Well Field, Transmission, and Distribution Reports (WTD) for fiscal year 2000 to present. The WTD reports are monthly reports that contain data related to, among other things, the well field pumpage. Well field pumpage is the water delivered to the treatment plant by the wells and is measured by four raw water meters at the head of the treatment plant. The monthly average day pumpage for fiscal years 2000 to 2012 is presented in Figure 3-5.



Source: Well Field, Transmission and Distribution Reports, 1999 to present.

Figure 3-5 City's Well Field Pumpage, Monthly Average 2000-2012

The SP production is the total of the peak three months of well field pumpage each year. The typical SP occurs in the June-July-August or July-August-September time frame. The SP is used to evaluate future well field production needs. A summary of the City's well field pumpage and SP is presented in Table 3-1. The highest seasonal peak/average day (SP:AD) ratio occurred in 2012 with a value of 1.56. The SP:AD ratio is the average day during the peak three months divided by the average day for the year. This maximum SP is rounded up to 1.6 for use as one of the planning criteria for assessing the water supply needs.

Table 3-1 City's Well Field Pumpage and Seasonal Peak Production

Fiscal Year	Well Field Pumpage Total (MG)	Well Field Pumpage Average (MGD)	Seasonal Peak Production Total (MG)	Seasonal Peak Production Average Day (AD) (MGD	Seasonal Peak Production Time Period (months)	SP/AD
2000	15,041	41.10	5,004	54.39	J,A,S [1]	1.32
2001	14,569	39.92	5,322	57.85	J,A,S [1]	1.45
2002[3]	15,122	41.43	5,884	63.96	J, J, A [2]	1.54
2003	14,513	39.76	5,491	59.68	J,A,S [1]	1.50
2004	13,885	37.94	4,604	50.04	J,A,S [1]	1.32
2005	14,775	40.48	5,558	60.41	J,A,S [1]	1.49
2006	14,851	40.69	5,240	56.96	J, J, A [2]	1.40
2007	13,369	36.63	5,180	56.30	J,A,S [1]	1.54
2008	12,906	35.26	4,371	47.51	J,A,S [1]	1.35
2009	12,512	34.28	4,068	44.22	J,A,S [1]	1.29
2010	12,062	33.05	4,448	48.35	J,A,S [1]	1.46
2011	13,111	35.92	4,675	50.82	J,A,S [1]	1.41
2012[3]	15,474	42.28	6,058	65.85	J, J, A [2]	1.56
					Planning Criteria	**1.6**

Source: Well Field, Transmission and Distribution Reports, fiscal year 2000 to present.
Notes:
1. *July, August, September*
2. *June, July, August*
3. *Water Restrictions*

3.4 Treatment and Transmission Usage

The WTD reports also provide the Lincoln usage, which is total water transmitted to the distribution system from the Platte River Water Treatment Facility (WTP) and the various pump stations and reservoirs in Lincoln. A comparison of the Lincoln usage and well field pumpage can be used to evaluate the quantity of water used for water treatment and transmission.

The treatment and transmission usage is determined by subtracting the Lincoln usage from the total well field pumpage reported on the WTD reports. This value accounts for the water losses or uses in the raw water transmission system, uses at the WTP (such as filter backwashing) and water losses or uses in the finished water transmission system between the WTP and the distribution system. A summary of the data is shown in Table 3-2.

Table 3-2 Treatment and Transmission Usage

Fiscal Year	Total Annual Well Field Pumpage (MG)	Total Annual Lincoln Usage (MG)	Total Annual Treatment and Transmission Usage (MG)	(%)
2000	15,041	15,265	-224	-1.5 [1]
2001	14,569	14,603	-34	-0.2 [1]
2002	15,122	14,807	315	2.1[1]
2003	14,513	13,693	820	6.0
2004	13,885	12,820	1065	8.3
2005	14,775	13,845	930	6.7
2006	14,851	14,025	826	5.9
2007	13,369	12,796	573	4.5
2008	13,006	11,984	1022	8.5
2009	12,512	11,941	571	4.8
2010	12,062	11,338	724	6.4
2011	13,111	11,686	1425	12.2
2012	15,474	14,032	1442	10.3
Average 2000-2012	**14,022**	**13,295**	**727**	**7.4**

Source: Well Field, Transmission, and Distribution Reports for fiscal year 2000 to present.
Note:
1. *Data from 2000 and 2001 are not included in determination of average due to apparent errors in data.*

Fiscal years 2011 and 2012 have higher percentage of treatment and transmission water usage than previous years. This higher percentage of plant water use is believed to be a result of issues the LWS has had in recent years with iron bacteria in the raw water transmission system. The bacteria required more frequent and extended filter backwash times and has resulted in shorter filter run times during spring events. In 2011 and 2012 filter backwashing was also increased due to higher manganese levels. The LWS is currently in the process of implementing improvements to minimize issues with the iron bacteria and as a result, it is anticipated that the percentage of water used for treatment and transmission usage should decline to a level similar to that experienced prior to 2011. The improvements being implemented will be further discussed in *Chapter 4 - Water Treatment* of the Master Plan.

3.5 Distribution System Usage

As discussed above, the WTD reports provide the Lincoln usage, which is the total water transmitted to the distribution system from the Platte River WTP and the various pump stations and reservoirs in Lincoln. This usage will be used to assess high service pumping, finished water transmission and distribution system needs. In addition, the Lincoln usage will be used to assess maximum day and maximum hour needs.

3.5.1 Historic System Wide Usage

The WTD reports provide the daily Lincoln usage and maximum hour Lincoln usage. Table 3-3 summarizes this data for fiscal year 2000 through fiscal year 2012. Based on this data, the ratios of maximum day to average day demands (MDD:ADD), maximum hour to average day demands (MHD:ADD) and maximum hour to maximum day demands (MHD:MDD) were calculated for this same period of time.

Table 3-3 Historic Water Usage and Peaking Factors

Fiscal Year	Total Annual Pumpage (BG)	Total Annual Lincoln Usage (BG)	Average Day Demand (ADD) (MGD)	Maximum Day Demand (MDD) (MGD)	Maximum Hour Demand (MHD) (MGD)	MDD:ADD	MHD:ADD	MHD:MDD
2000	15.0	15.3	41.8	86.0	127.5	2.1	3.1	1.5
2001	14.6	14.6	40.0	85.5	102.1	2.1	2.6	1.2
2002	15.1	14.8	40.5	90.4	136.9	2.2	3.4	1.5
2003	14.5	13.7	37.5	78.0	125.7	2.1	3.4	1.6
2004	13.9	12.8	35.0	65.8	93.3	1.9	2.7	1.4
2005	14.8	13.8	37.8	87.6	114.1	2.3	3.0	1.3
2006	14.9	14.0	38.4	75.7	117.6	2.0	3.1	1.6
2007	13.4	12.8	35.1	84.9	122.6	2.4	3.5	1.4
2008	13.0	12.0	32.7	69.1	117.7	2.1	3.6	1.7
2009	12.5	11.9	32.7	60.1	136.7	1.8	4.2	2.3
2010	12.0	11.3	31.1	70.1	133.3	2.3	4.3	1.9
2011	13.1	11.7	32.0	69.3	127.5	2.2	4.0	1.8
2012[1]	15.5	14.0	38.4	80.0	173.0	2.1	4.5	2.2
Average 2000-2012	**14.0**	**13.3**	**36.4**	**77.1**	**125.2**	**2.1**	**3.4**	**1.6**

Source: Well Field, Transmission and Distribution Reports, fiscal year 2000 to fiscal year 2012
Note:
1 Water restrictions were implemented from July through September 2012.

3.5.2 System Maximum Day and Maximum Hour Demands

In the development of demand projections, it is important that the water system be capable of meeting the needs of its customers for both the maximum day and maximum hour conditions. Variations in these demand conditions will occur year to year based on factors such as precipitation and temperature. If the peak demands exceed the capacity of the system, water restrictions would need to be imposed.

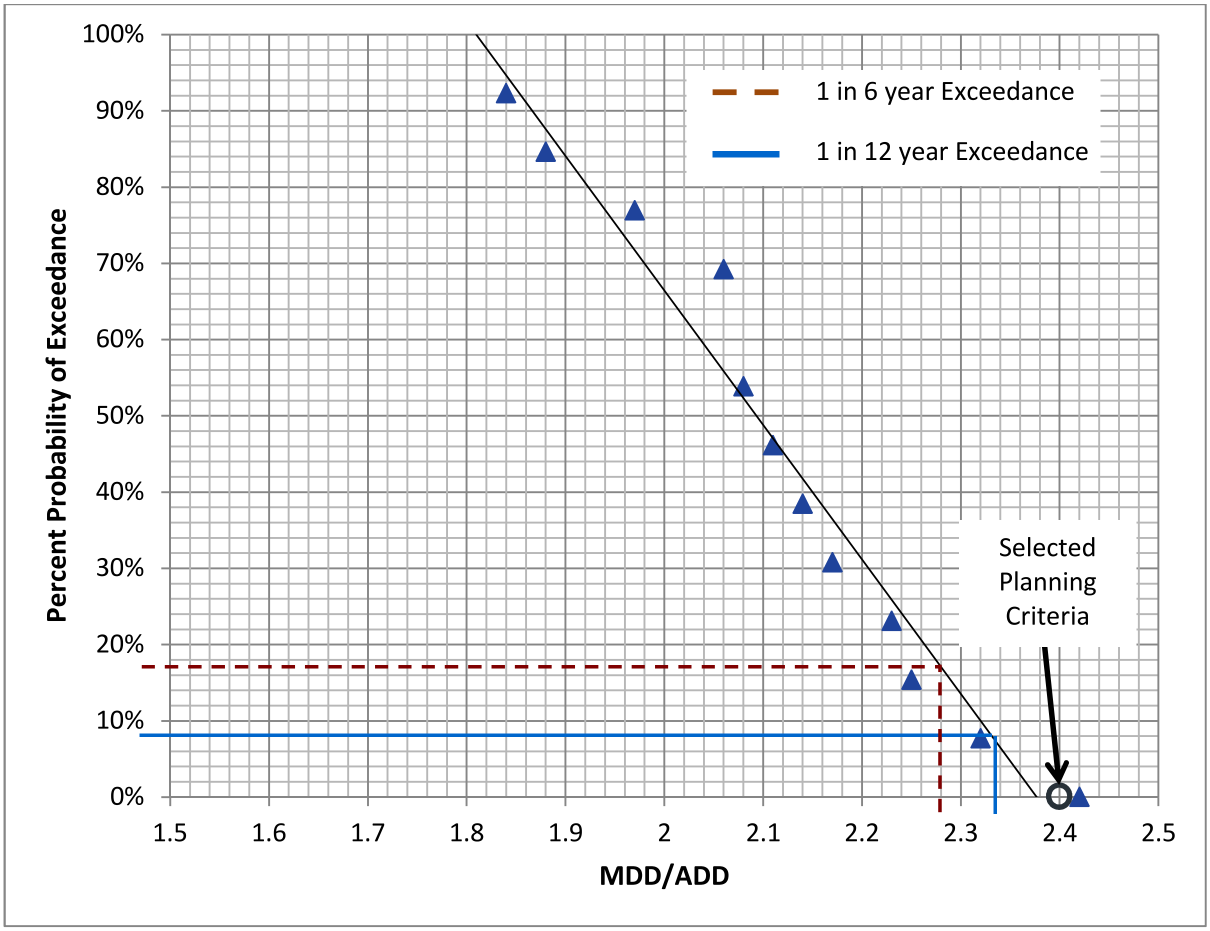
The assessment of a reasonable level of peaking factors for future projections of maximum day and MHD includes a careful evaluation of the desired level of system reliability and the cost of implementing that level of reliability. If too low of a peaking factor is selected, the reoccurrence of required water restrictions would exceed the acceptable levels for the community. If too high of a peaking factor is selected, the costs of implementing the infrastructure required to support this system capacity would exceed the financial resources of the LWS.

To achieve the required balance between these two factors, an analysis of the acceptable return period for historical peaking factors is required. The Recommended Standards for Water Works 2012 Edition (Ten States Standards) recommends a 1-in-50 year drought reoccurrence interval for planning of surface water supplies. A similar recommendation is not made for planning of groundwater supplies. As a result, the selection of the acceptable level of reoccurrence of drought or historical peaking factors must be made on a system-by-system basis with consideration of the acceptable level of reliability and the resulting costs for the community.

In the 2002 Master Plan, a reoccurrence interval of 1-in-12 years was selected for the LWS. This correlates to approximately an 8th percentile of the probability of exceedance for the maximum day and MHD based on the historical period of record considered in the evaluation. An evaluation of this percentile of exceedance was conducted for the ratio of peak demands to ADD or peaking factor, based upon the period of record from fiscal year 2000 to fiscal year 2012.

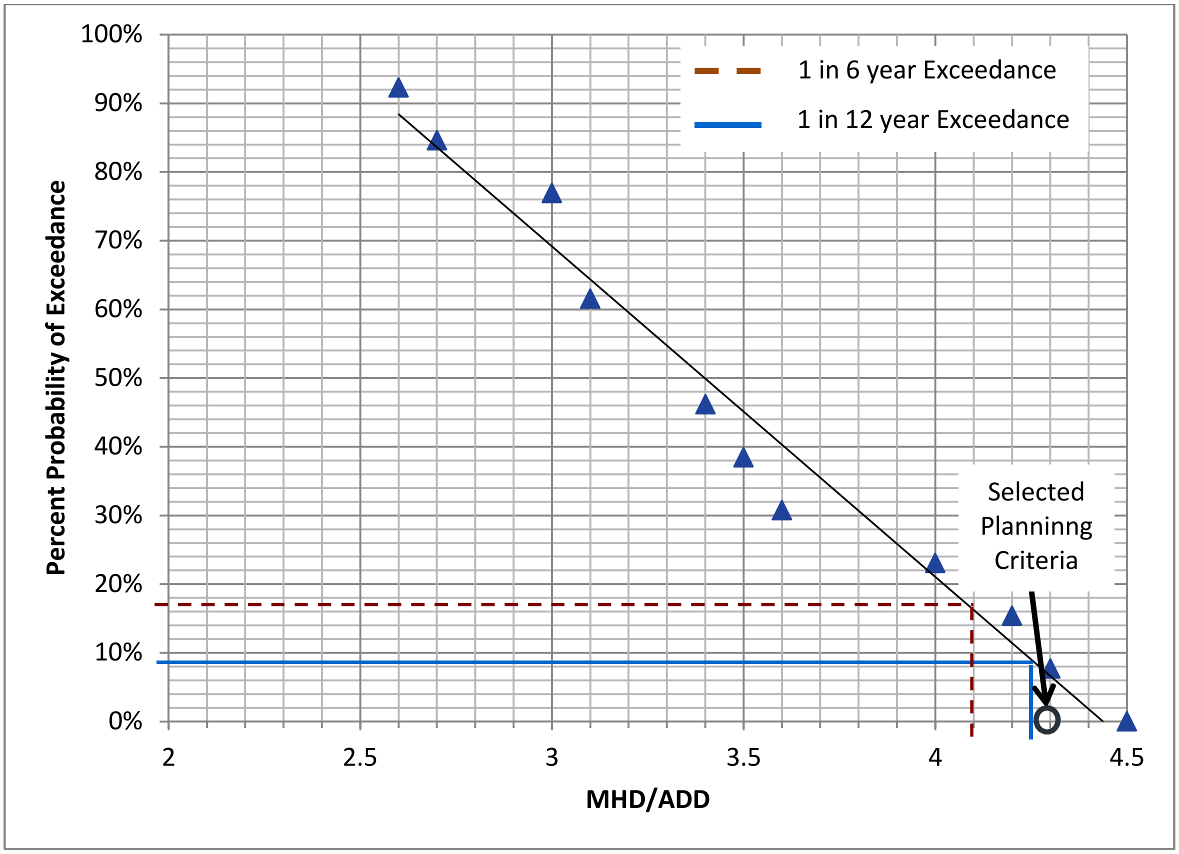
Based upon discussions with the City, a 16th percentile of the probability of exceedance was also evaluated to determine the potential cost savings associated with a less conservative reoccurrence interval. A 16th percentile of the probability of exceedance corresponds to a 1-in-6 year reoccurrence interval for exceeding the maximum day and MHD which could result in water use restrictions.

Figure 3-6 presents the frequency distribution for MDD:ADD peaking factors and Figure 3-7 presents the frequency distribution for the MHD:ADD peaking factors based upon the historical peaking factors presented in Table 3-3.



Note:
Peaking factors plotted are from Table 3-3 for fiscal year 2000 to fiscal year 2012.

Figure 3-6 MDD:ADD, Percent Probability of Exceedance



Note:
Peaking factors plotted are from Table 3-3 for fiscal year 2000 to fiscal year 2012.

Figure 3-7 MHD:ADD, Percent Probability of Exceedance

As shown on Figures 3-6 and 3-7, the difference between the 1-in-12 year reoccurrence interval and the 1-in-6 year reoccurrence interval was not a substantial change in the selected peaking factors. With respect to the timing of CIP work; the 1-in-12 year reoccurrence interval moved projects forward 2-3 years vs. using the 1-in-6 reoccurrence interval, which is not significant. As a result, the more conservative level of reliability, 1-in-12 year reoccurrence, was selected for the basis of developing the demand projections.

This analysis resulted in a MDD:ADD peaking factor of 2.4 and a MHD:ADD peaking factor of 4.3. As a sensitivity analysis, the same frequency distribution was conducted for a 20-year historical period of record (1993-2012). For the 20-year frequency distribution, the MDD:ADD peaking factor was calculated to be 2.4 and the MHD:ADD peaking factor was calculated to be 4.1 for a 1-in-12 year reoccurrence interval. As a result, it was determined that the use of the period of 2000 to 2012 provided a reasonable basis for the frequency distribution analysis. In summary, the peaking factors that will be used as planning criteria for the demand projections are as follows:

- MDD:ADD peaking factor – 2.4

- MHD:ADD peaking factor – 4.3

3.5.3 Service Level Maximum Day and Maximum Hour Peaking Factors

Maximum day and maximum hour usage projections for each service level were evaluated based on the peaking factors used in the 2007 Master Plan and 2012 Supervisory Control and Data Acquisition (SCADA) data provided by the City. The SCADA data was analyzed to determine the ADD from May 2012 (close to 2012 ADD), the MDD from July 24, 2012, and the MHD from August 17, 2012 for each service level. The demands by service level were established by subtracting hour volumes of service level demand sources (exiting pump stations and transfers) from hour volumes of service level supply sources (entering pump stations and transfers) in each service level. The change in storage reservoir levels was also incorporated by calculating the hourly volume by level change with a negative change in volume representing a demand source and a positive change in volume representing a supply source. The remaining volume from this calculation is equal to the service level demand.

Individual service level maximum day peaking factors were calculated from the resulting July MDD compared to the May ADD. Individual service level maximum hour peaking factors were calculated from the resulting August MHD compared to the May ADD peaking factors by class within each service level were adjusted slightly from the 2007 Master Plan using the calculated 2012 MDD and MHD peaking factors so that the sum of usage by service level would match the total system usage projections and system-wide peaking factors. Planning peaking factors by class and service level are summarized in Table 3-4.

Table 3-4 Planning Peaking Factors by Class and Service Level

Service Level	2012 Calculated MDD PF	2012 Calculated MHD PF	Residential Peaking Factors								Commercial Peaking Factors	
			Maximum Day				Maximum Hour					
			Base	2025	2040	2060	Base	2025	2040	2060	All Years MD	All Years MH
Northwest	3.95	5.25	6.3	5.0	4.2	3.4	7.5	6.8	6.5	6.0	1.8	4.0
Belmont	2.01	2.48[1]	2.0	2.2	2.3	2.4	4.4	4.0	4.0	3.8	1.8	3.3
Low	2.95	4.15	3.0	3.3	3.3	3.3	4.4	3.9	3.9	3.8	2.0	4.2
High	2.00	2.75[1]	2.0	2.6	2.6	2.6	5.4	4.8	4.8	4.8	1.8	3.7
Southeast	2.42	9.37[2]	2.4	3.0	3.0	3.0	6.4	5.5	5.5	5.5	2.2	5.4
Cheney	2.15	4.63	2.1	2.6	2.7	2.7	6.4	5.6	5.6	5.5	2.0	4.6
Overall	**2.1**	**4.5**	**2.3**	**2.8**	**2.8**	**2.8**	**5.3**	**4.8**	**4.8**	**4.8**	**2.0**	**4.0**

Source: 2012 SCADA data and overall system peaking factors.

Notes:

1. *Non-revenue water peaking factor = 1.0 for all conditions and all planning years.*
2. *MHD peaking factors for Belmont and High service levels appear low compared to the other MHD peaking factors, so they were not used as the basis of existing residential and commercial peaking factors.*
3. *MHD peaking factor for Southeast appears high compared to the other MHD peaking factors so it was not used as the basis of existing*

3.6 Historic Metered Sales

3.6.1 Historic System Metered Sales

Historic metered sales were obtained from the LWS billing information for the period of fiscal year 2000 to fiscal year 2012. This data was used to assess average per capita demands and the distribution of water demands between residential and non-residential users. In addition, this data was used to evaluate non-revenue water (NRW), which is water that is used for flushing hydrants, fire fighting, water main breaks, leaks and other maintenance activities and is calculated as the difference between the Lincoln usage and the metered sales.

Table 3-5 summarizes the historical metered sales, including the distribution between residential and non-residential users; average day Lincoln usage; and the total percentage for each year of NRW.

As presented in Table 3-5, the planning criteria that will be used for the Master Plan include an average residential demand of approximately 65 percent of the total system demand and an average NRW of 6.7 percent of the average system demand. Table 3-6 and Figure 3-8 present results of the analysis of the gallons per capita per day (gpcd) usage and is broken into residential, total metered sales, and Lincoln usage. The per capita values were obtained by dividing average day values for each year by the population for that year.

Table 3-5 Historic Metered Sales

Fiscal Year	Historical Metered Sales					Average Day Lincoln Usage	Non-Revenue Water (NRW)	Non-Revenue Water 10-yr Running Average
	Residential		Non-Residential		Total			
	(MGD)	(%)	(MGD)	(%)	(MGD)	(MGD)	(% of AD)	(% of AD)
1987					29.3	31.5	7.0	
1988	20.6	62	12.4	38	33.0	35.3	6.5	
1989	22.0	63	12.8	37	34.8	35.6	2.2	
1990	18.9	61	12.0	39	30.9	32.8	5.8	
1991	20.2	62	12.3	38	32.5	34.6	6.1	
1992	17.7	61	11.2	39	28.9	31.8	9.1	
1993	16.0	60	10.5	40	26.5	28.9	8.3	
1994	18.0	61	11.3	38	29.4	31.0	5.2	
1995	20.1	63	11.9	37	32.0	34.2	6.4	
1996	19.0	62	11.7	38	30.7	33.2	7.5	6.41
1997	20.2	62	12.5	38	32.7	34.7	5.8	6.29
1998	19.6	61	12.5	39	32.1	34.5	7.0	6.34
1999	19.3	61	12.4	39	31.7	34.7	8.6	6.98
2000	23.7	65	12.9	35	36.6	41.2	11.2	7.52
2001	21.8	63	12.7	37	34.5	39.1	11.8	8.09
2002	23.9	65	12.8	35	36.7	39.7	7.6	7.94
2003	22.3	65	11.9	35	34.2	37.5	8.8	7.99
2004	22.2	65	11.9	35	34.1	35.0	2.6	7.73
2005	23.9	67	11.9	33	35.8	38.5	7.0	7.79
2006	24.1	66	12.2	34	36.3	36.5	0.5	7.09
2007	21.5	65	11.7	35	33.2	35.1	5.4	7.05
2008	19.6	64	10.8	36	30.4	32.7	7.0	7.05
2009	20.8	67	10.3	33	31.1	32.7	4.9	6.68
2010	18.9	66	9.7	34	28.6	31.1	8.0	6.36
2011	20.9	67	10.5	33	31.4	32.0	1.9	5.37
2012	22.8	66	11.7	34	34.5	38.4	10.2	5.63
Average 2000-2012	**22.0**	**65**	**11.6**	**35**	**33.6**	**36.1**	**6.7**	**6.96**

Source: Data from years 1987-1999 were obtained from 2002 Master Plan. This data is presented for reference only, and is not used as part of analysis. Data from years 2000-2012 were obtained from LWS billing data.
Note:
Population for years 2000 and 2010 was determined from linear interpolation between the census data in 2000 and 2010.

Table 3-6 Historic Per Capita Usage

Fiscal Year	Population	Residential Sales		Total Metered Sales		Average Day Lincoln Usage	
		Total (MGD)	Per-Capita (gpcd)	Total (MGD)	Per-Capita (gpcd)	Total (MGD)	Per-Capita (gpcd)
1987	185,960	-	-	29.3	158	31.5	169
1988	187,964	20.6	110	33.0	176	35.3	188
1989	189,968	22.0	116	34.8	183	35.6	187
1990	191,972	18.9	98	30.9	161	32.8	171
1991	195,333	20.2	103	32.5	166	34.6	177
1992	198,694	17.7	89	28.9	145	31.8	160
1993	202,055	16.0	79	26.5	131	28.9	143
1994	205,416	18.0	88	29.4	143	31.0	151
1995	208,777	20.1	96	32.0	153	34.2	164
1996	212,137	19.0	90	30.7	145	33.2	157
1997	215,498	20.2	94	32.7	152	34.7	161
1998	218,859	19.6	90	32.1	147	34.5	158
1999	222,220	19.3	87	31.7	143	34.7	156
2000	225,581	23.7	105	36.6	162	41.2	183
2001	228,861	21.8	95	34.5	151	39.1	171
2002	232,141	23.9	103	36.7	158	39.7	171
2003	235,421	22.3	95	34.2	145	37.5	159
2004	238,701	22.2	93	34.1	143	35.0	147
2005	241,981	23.9	99	35.8	148	38.5	159
2006	245,261	24.1	98	36.3	148	36.5	149
2007	248,541	21.5	87	33.2	134	35.1	141
2008	251,821	19.6	78	30.4	121	32.7	130
2009	255,101	20.8	82	31.1	122	32.7	128
2010	258,379	18.9	73	28.6	111	31.1	120
2011	261,480	20.9	80	31.4	120	32.0	122
2012	264,618	22.8	86	34.5	130	38.4	145
Ave. 2000-2012	**245,222**	**22**	**90**	**33.6**	**138**	**36.1**	**148**

Source: Data from years 1987-1999 were obtained from 2002 Master Plan. This data is presented for reference only, and is not used as part of analysis. Data from years 2000-2012 were obtained from LWS billing data.

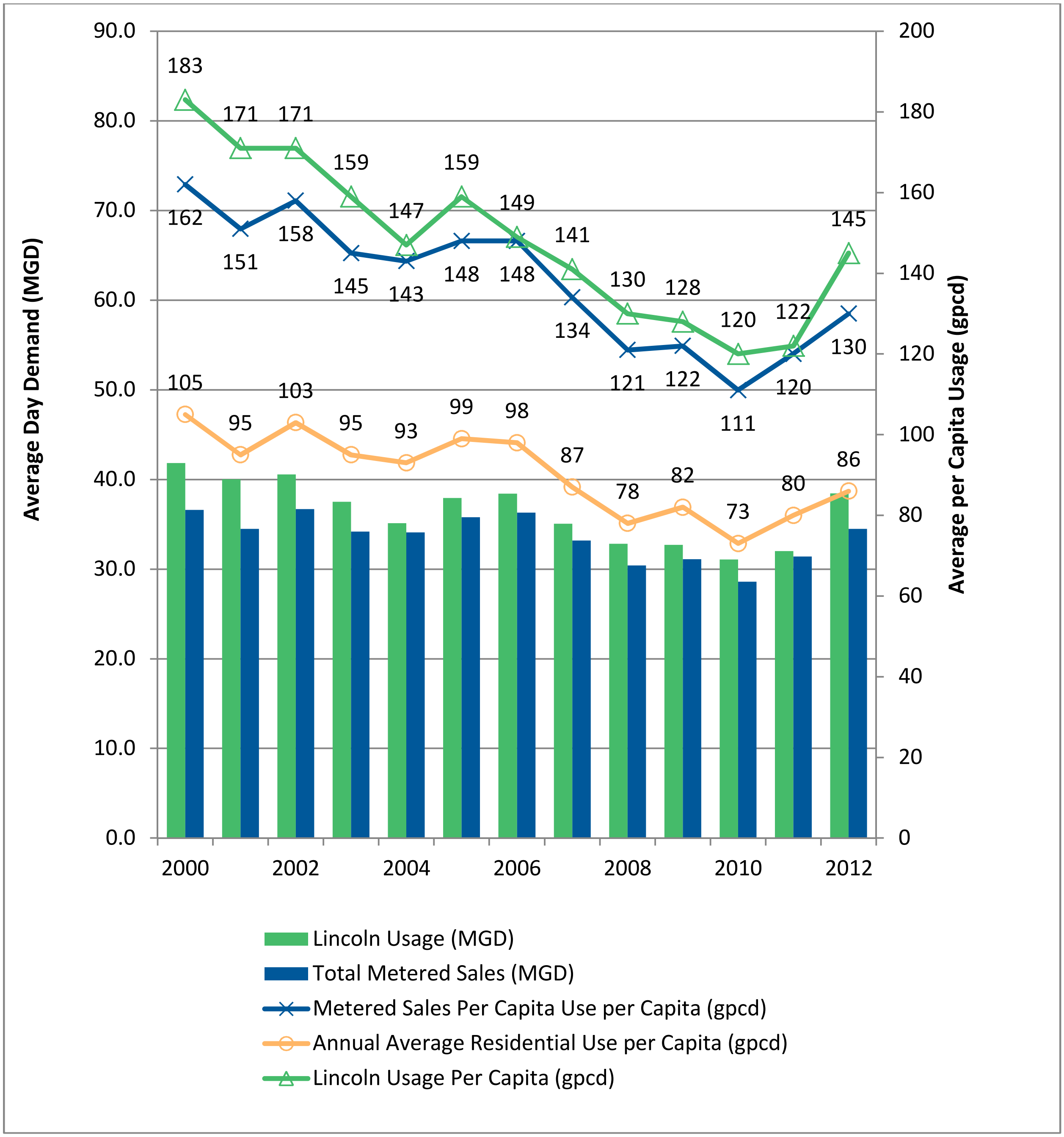


Figure 3-8 Historic Water Usage

3.6.2 Historic Metered Sales by Service Level

The City provided fiscal year 2012 geocoded meter sales data from the Billing Department for every account in the LWS distribution system, consisting of approximately 82,800 records. The information included meter ID, account number, account address, service address, bi-monthly metered units, and a user classification code (residential or non-residential). Initially, the GIS meter locations were matched to the meter data records using the meter IDs. Due to differences between GIS locations and the billing data records, there is not a one-to-one match between the two data sources. To match additional meter sales between the billing meter data and the GIS locations, the following steps were taken:

1. Matching GIS meter service addresses to billing addresses.
2. Matching GIS meter account numbers to billing account numbers.
3. Matching significant users (over 10,000 gpd) in the billing records to GIS meters by manually mapping billing addresses to determine if they were likely service addresses and assigning it to the nearest GIS meter.

The process resulted in a 97% match in total meter locations (80,688 of 82,821) and in total water sales volume (35.2 of 36.2 mgd) which was determined to be acceptable based on experience with other planning efforts and a similar analysis conducted in the 2007 Master Plan. The total 2012 metered sales were then summarized by service level and residential or non-residential use as shown in Table 3-7 by overlaying the meters with the existing service levels in GIS.

Table 3-7 Year 2012 Metered Sales by Service Level

Service Level	Fiscal Year 2012 Metered Sales (mgd)		
	Residential	Non-Residential	Total
Northwest	0.3	0.2	0.5
Belmont	3.2	1.2	4.4
Low	4.6	5.6	10.2
High	9.2	3.6	12.8
Southeast	4.8	0.9	5.7
Cheney	0.7	0.2	0.9
Total	**22.8**	**11.7**	**34.5**

Source: Geocoded meter data and GIS meters.

The estimated population served in each service level was compared against the residential metered sales in each service level to calculate the residential per-capita water use by service level for the year 2012, as shown in Table 3-8. For comparison, the historic residential per-capita water use from 2006 is also shown in the table. Since 2006, there has been an overall reduction of residential per-capita demand in all service levels, which is consistent with the system-wide analysis.

Table 3-8 Year 2012 Per-capita Residential Use by Service Level

Service Level	Fiscal Year 2012 Metered Sales			FY 2006 Meter Sales
	Population	Residential Sales (mgd)	Residential Per-capita Use (gpcd)	Residential Per-capita Use (gpcd)
Northwest	2,299	0.3	130	170
Belmont	40,922	3.2	78	101
Low	76,760	4.6	60	67
High	102,265	9.2	90	92
Southeast	38,382	4.8	125	151
Cheney	3,990	0.7	175	211
Total System	**264,618**	**22.8**	**86**	**94**

Source: Population estimates for 2012, geocoded meter data and GIS meters.

3.7 Water Demand Projections

3.7.1 System Demand Projections

System demand projections have been developed for the design years 2025, 2040, and 2060, as well as decennial years in the planning horizon. The demand projections are based on the population forecasts, the per-capita demand, and the peaking factors for the maximum day and the maximum hour previously presented. Table 3-9 summarizes the system-wide planning criteria used for the demand projections. For comparison purposes, the similar criteria that were developed in the 2007 Master Plan are also presented in the tables below.

Table 3-9 Average Day Demand Design Criteria

	2007 Master Plan	2013 Master Plan
Per-capita Residential Metered Sales (gal/day)	96	90 [1]
Residential Sales as Percent of Total Metered Sales	65%	65% [2]
Per-capita Total Metered Sales (gal/day)	148	138
Non-Revenue Water (Percent of Lincoln Usage)	6.25%	6.7%
Total Lincoln Usage as Per-capita Usage (gal/day)	157	148
Transmission and Treatment Uses (Percent of Lincoln Usage)	3%	6.9%
Transmission and Treatment Uses (gpcd)	5	10
Well field Pumpage (gpcd) (Demand)	162	158

Notes:

1. *From Table 3-6, average residential per capita sales based on fiscal year 2000 to fiscal year 2012*
2. *From Table 3-5, average percentage of residential sales based on fiscal year 2000 to fiscal year 2012*

Table 3-10 Maximum Day and Maximum Hour Design Criteria

	2007 Master Plan	2013 Master Plan
MDD:ADD Peaking Factor	2.7	2.4 [1]
MHD:ADD Peaking Factor	4.4	4.3 [1]
Maximum Day Demand (gpcd) (Lincoln Usage)	437	379
Maximum Hour Demand (gpcd) (Lincoln Usage)	693	636

Note:

1. Based on 8th percentile of the probability of exceedance from the historical period of fiscal year 2000 to fiscal year 2012

Table 3-11 provides a summary of the demand projections based on the planning criteria for the planning years 2025, 2040 and 2060 as well as the decennial years in the planning horizon. Demand projections were developed for well field pumpage for average day and maximum day. Projections for Lincoln usage were developed for the average day, maximum day, and maximum hour conditions. Figure 3-9 graphically depicts the demand projections.

Table 3-11 Future Demand Projections

Year	Estimated Population	Average Day Well Field Pumpage (MGD)	Average Day Lincoln Usage (MGD)	Maximum Day Well Field Pumpage (MGD)	Maximum Day Lincoln Usage (MGD)	Maximum Hour Lincoln Usage (MGD)
2012	264,618	42.4	38.4	83.3	80	173
2020	291,100	46	43	110	103	185
2025	309,000	49	46	117	110	197
2030	328,000	52	49	124	116	209
2040	371,700	59	55	141	132	237
2050	416,400	66	62	158	148	265
2060	461,700	73	68	175	164	294

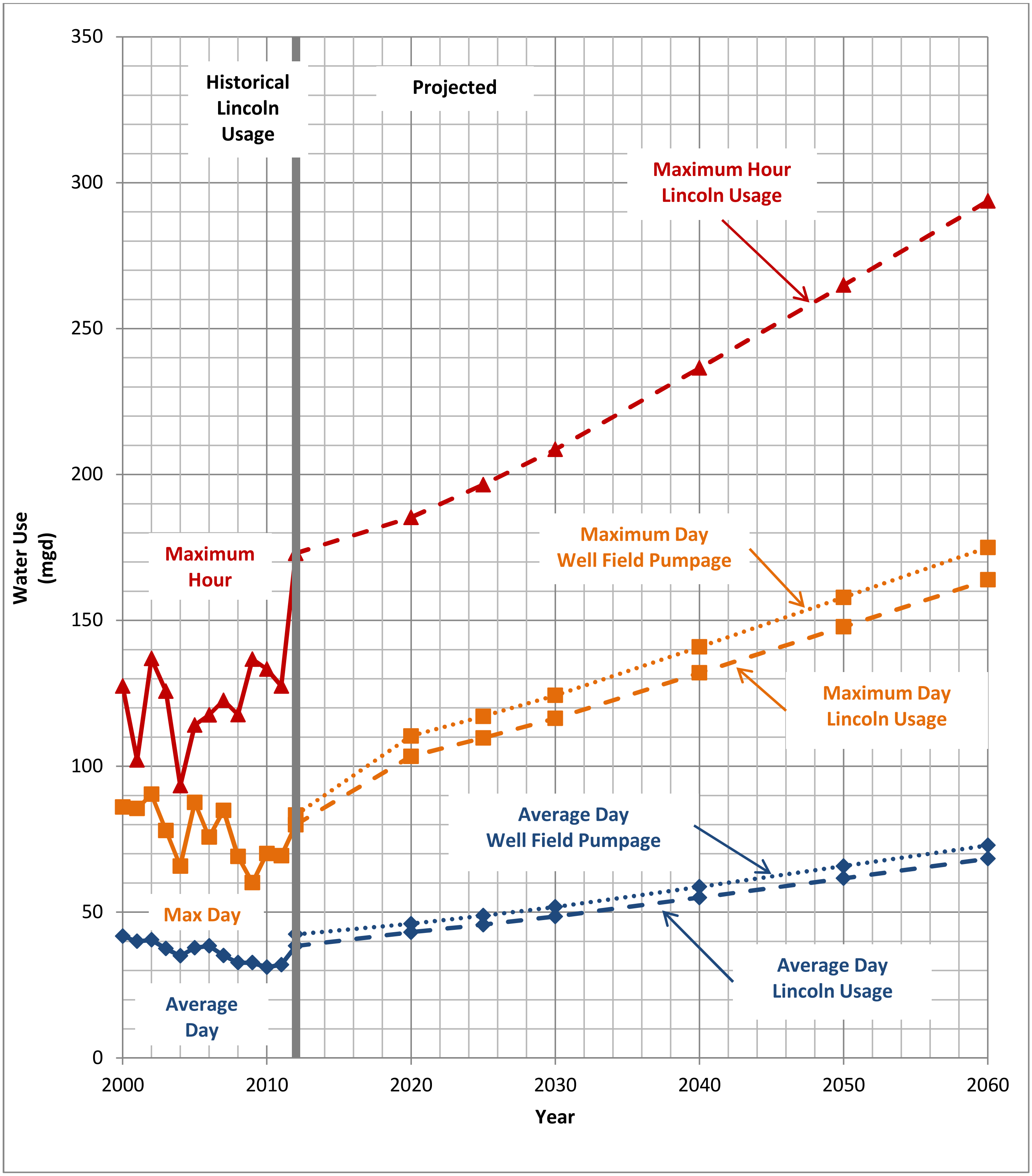


Figure 3-9 Future Demand Projections

3.7.2 Water Usage Projections by Service Level

Based on the total system usage projections, base year (2012), and future years 2025, 2040, and 2060 average day usage by service level were determined as shown in Table 3-12. Figure 3-10 depicts the average day usage projections from 2012 through 2060 by service level.

The base year sales by service level was based on the 2012 meter sales for both residential and non-residential usage classes and the existing service level boundaries. The calculated 10.2% system-wide NRW volume during 2012 was added to the water sales by service level to arrive at the base year average day usage by service level.

The 2025, 2040, and 2060 water sales by service level were based on three factors:

- To capture population-driven usage in infill and redevelopment areas, the population increases over 2012 in each TAZ were multiplied by the service level per capita usage and added to the existing 2012 meter usage.
- To capture population-driven usage in future extension areas, the population in each TAZ was multiplied by the service level per capita usage.
- To capture commercial and industrial usage changes in infill, redevelopment and future extension areas, total commercial and industrial areas provided by Lincoln-Lancaster County Planning Department between 2010 and 2040 were used to re-adjust the residential versus non-residential percent usage between planning years.

For each of the future planning years, a balance between 90 gpcd per-capita residential meter sales, 65% residential sales as a percent of total metered sales, and 138 gpcd total metered sales was established on a system-wide level to match the ADD planning criteria. Individual service level per-capita residential meter sales and residential sales as a percent of total metered sales were adjusted to meet the system-wide demand planning criteria based on the three factors listed previously and the expected continuation of decreased per-capita residential usage. The 6.7% NRW allocation as a percent of Lincoln usage was added to the meter sales to establish usage by service level.

The base year and planning years 2025, 2040, and 2060 usage projections by service level were allocated and will be used for hydraulic model development, calibration and analysis.

Table 3-12 Average Day Usage Projections by Class and Service Level

Service Level	Residential Per-capita Sales (gpcd)	Residential Sales (MGD)	Residential/ Total Sales (%)	Non Residential Sales (MGD)	Total Sales (MGD)	NRW (%)	NRW (MGD)	Average Day Usage (MGD)
Base Year (2012)								
Northwest	130	0.3	60%	0.2	0.5	10.2%	0.1	0.6
Belmont	78	3.2	73%	1.2	4.4	10.2%	0.5	4.9
Low	60	4.6	45%	5.6	10.2	10.2%	1.2	11.4
High	90	9.2	72%	3.6	12.8	10.2%	1.4	14.2
Southeast	125	4.8	84%	0.9	5.7	10.2%	0.6	6.3
Cheney	175	0.7	78%	0.2	0.9	10.2%	0.1	1.0
Total	**86**	**22.8**	**66%**	**11.7**	**34.5**	**10.2%**	**3.9**	**38.4**
Planning Year 2025								
Northwest	133	0.7	52%	0.6	1.3	6.7%	0.1	1.4
Belmont	77	4.0	71%	1.7	5.7	6.7%	0.4	6.1
Low	60	4.6	43%	6.1	10.7	6.7%	0.7	11.4
High	88	10.0	71%	4.1	14.1	6.7%	1.0	15.1
Southeast	128	6.1	82%	1.3	7.4	6.7%	0.5	7.9
Cheney	182	2.5	72%	1.0	3.5	6.7%	0.3	3.8
Total	**90**	**27.9**	**65%**	**14.8**	**42.7**	**6.7%**	**3.0**	**45.7**
Planning Year 2040								
Northwest	132	0.9	50%	0.8	1.7	6.7%	0.1	1.8
Belmont	76	4.9	67%	2.4	7.3	6.7%	0.5	7.8
Low	56	5.0	43%	6.7	11.7	6.7%	0.8	12.5
High	87	11.0	71%	4.5	15.5	6.7%	1.1	16.6
Southeast	127	8.4	81%	2.0	10.4	6.7%	0.8	11.2
Cheney	180	3.3	70%	1.5	4.8	6.7%	0.3	5.1
Total	**90**	**33.5**	**65%**	**17.8**	**51.3**	**6.7%**	**3.6**	**55.0**
Planning Year 2060								
Northwest	127	1.2	50%	1.2	2.4	6.7%	0.2	2.6
Belmont	74	6.1	66%	3.2	9.3	6.7%	0.7	10.0
Low	56	5.6	43%	7.5	13.1	6.7%	0.9	14.0
High	85	12.5	71%	5.1	17.6	6.7%	1.3	18.9
Southeast	124	11.9	79%	3.2	15.1	6.7%	1.1	16.2
Cheney	168	4.2	68%	2.0	6.2	6.7%	0.4	6.6
Total	**90**	**41.5**	**65%**	**22.2**	**63.7**	**6.7%**	**4.6**	**68.3**

Note:
Usage includes non-revenue water estimates; however they do not include treatment and transmission usage.

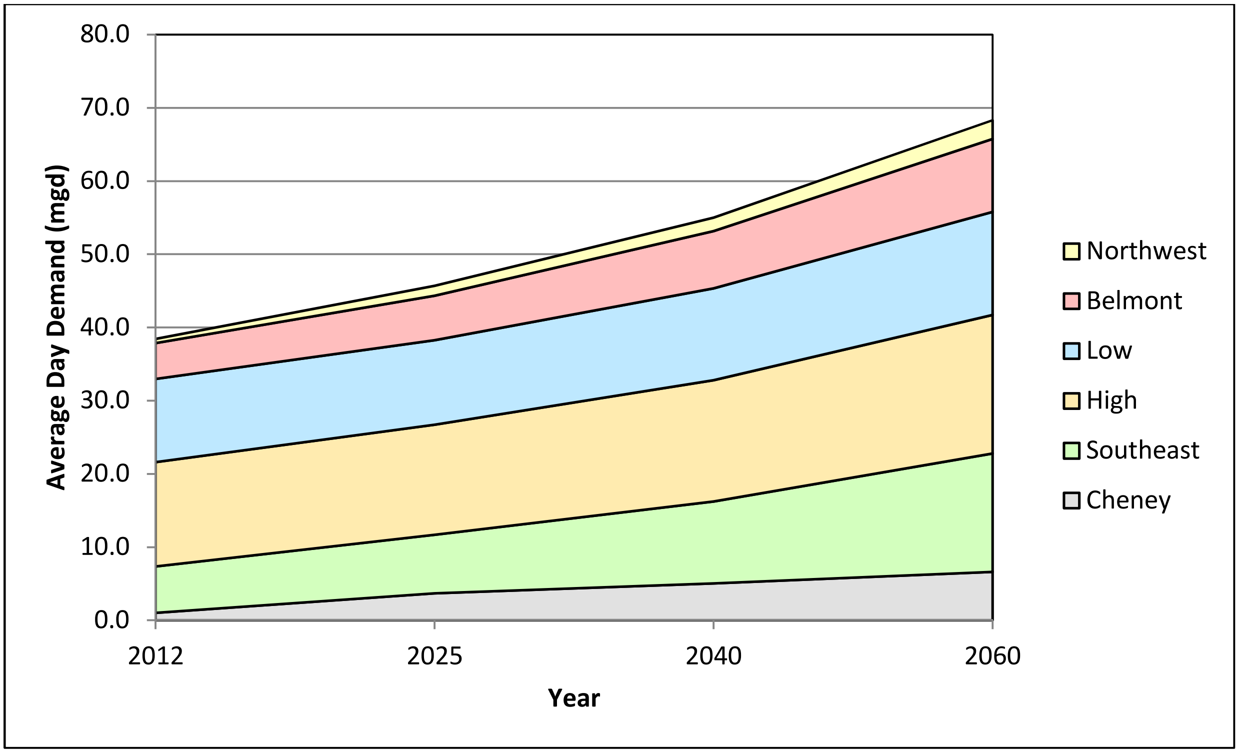


Figure 3-10 Future Average Day Usage Projections by Service Level

From the future average day usage projections by service level and the developed MDD and MHD peaking factors summarized in Table 3-4, the water usage projections by service level for ADD, MDD, and MHD were established. Table 3-13 presents the water usage projections by service level, including contributions from residential and non-residential customer classes for the base year (2012) and planning years 2025, 2040, and 2060.

Table 3-13 Water Usage Projections by Service Level

Service Level	Average Day			Maximum Day			Maximum Hour		
	Res. Usage (MGD)	Non-Res. Usage (MGD)	Total Usage (MGD)	Res. Usage (MGD)	Non-Res. Usage (MGD)	Total Usage (MGD)	Res. Usage (MGD)	Non-Res. Usage (MGD)	Total Usage (MGD)
Base Year (2012)									
Northwest	0.4	0.2	**0.6**	2.0	0.4	**2.4**	2.3	0.8	**3.2**
Belmont	3.6	1.3	**4.9**	6.6	2.3	**8.9**	14.3	4.1	**18.4**
Low	5.1	6.3	**11.4**	14.2	12.1	**26.3**	21.0	24.0	**44.9**
High	10.2	4.0	**14.2**	19.4	6.9	**26.3**	51.1	13.7	**64.8**
Southeast	5.3	1.0	**6.3**	12.1	2.1	**14.2**	31.2	5.0	**36.2**
Cheney	0.8	0.2	**1.0**	1.5	0.4	**1.9**	4.6	0.9	**5.5**
Total	**25.4**	**13.0**	**38.4**	**55.8**	**24.1**	**80.0**	**124.4**	**48.5**	**173.0**
Planning Year 2025									
Northwest	0.8	0.6	**1.4**	3.6	1.1	**4.7**	4.8	2.4	**7.3**
Belmont	4.3	1.8	**6.1**	9.2	3.2	**12.4**	16.3	5.8	**22.0**
Low	4.9	6.5	**11.4**	15.5	12.8	**28.3**	18.2	25.5	**43.7**
High	10.7	4.4	**15.1**	26.7	7.7	**34.4**	48.7	15.4	**64.1**
Southeast	6.5	1.4	**7.9**	18.5	2.9	**21.4**	34.1	7.1	**41.1**
Cheney	2.7	1.1	**3.8**	6.7	2.1	**8.8**	14.1	4.7	**18.7**
Total	**29.9**	**15.8**	**45.7**	**80.1**	**29.9**	**110.0**	**136.2**	**60.8**	**197.0**
Planning Year 2040									
Northwest	1.0	0.8	**1.8**	3.9	1.5	5.3	5.9	3.1	**9.0**
Belmont	5.2	2.6	**7.8**	11.4	4.5	15.9	19.7	8.0	**27.7**
Low	5.3	7.2	**12.5**	16.8	14.0	30.8	19.6	27.8	**47.4**
High	11.8	4.8	**16.6**	29.4	8.3	37.7	53.4	16.5	**69.9**
Southeast	9.0	2.2	**11.2**	25.5	4.5	30.0	46.8	10.7	**57.5**
Cheney	3.5	1.6	**5.1**	9.1	3.2	12.3	18.5	7.0	**25.5**
Total	**35.9**	**19.1**	**55.0**	**96.1**	**36.0**	**132.0**	**163.9**	**73.1**	**237.0**
Planning Year 2060									
Northwest	1.3	1.3	**2.6**	4.2	2.3	6.4	7.3	4.8	**12.1**
Belmont	6.6	3.4	**10.0**	14.9	6.1	21.0	23.7	10.5	**34.2**
Low	6.0	8.0	**14.0**	18.9	15.6	34.5	21.9	30.7	**52.6**
High	13.4	5.5	**18.9**	33.4	9.5	42.9	60.3	18.8	**79.1**
Southeast	12.8	3.4	**16.2**	36.1	7.3	43.4	66.3	17.3	**83.6**
Cheney	4.5	2.1	**6.6**	11.6	4.2	15.8	23.3	9.1	**32.4**
Total	**44.5**	**23.8**	**68.3**	**119.1**	**44.9**	**164.0**	**202.8**	**91.2**	**294.0**

Note:
Usage includes non-revenue water estimates; however they do not include treatment and transmission usage.

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CITY OF LINCOLN NEBRASKA

Lincoln Water System Facilities Master Plan

Chapter 3 - Water Supply

May 2014

HDR

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Abbreviations and Acronyms

ASR	aquifer storage and recovery
cfs	cubic feet per second
City	City of Lincoln, Nebraska
Dakota Formation	Cretaceous Dakota Aquifer
EIS	Environmental Impact Statement
EPA	U.S. Environmental Protection Agency
ESA	Endangered Species Act
fps	feet per second
GIS	geographic information system
gpm	gallons per minute
gpm/ft	gallons per minute per foot
HCW	horizontal collector well
HEC-SSP	Hydrologic Engineering Center Statistical Software Package
HDR	HDR Engineering, Inc.
LF	linear foot
LI	Langelier Index
LPSNRD	Lower Platte South Natural Resources District
LWS	Lincoln Water System
2002 Master Plan	2002 Facilities Master Plan
Master Plan	2013 Facilities Master Plan
mg/L	milligrams per liter
mgd	million gallons per day
MUD	Metropolitan Utilities District
NDEQ	Nebraska Department of Environmental Quality
NDNR	Nebraska Department of Natural Resources
NPDES	National Pollutant Discharge Elimination System
NRD	Natural Resources District
ppm	parts per million
psi	pounds per square inch

sq. mi.	square miles
TDS	total dissolved solids
UBBNRD	Upper Big Blue Natural Resources District
UIC	Underground Injection Control
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
WWTP	Wastewater Treatment Plant

1.0 Introduction

Chapter 3-Water Supply of the 2013 Facilities Master Plan (Master Plan) establishes the current capacity of Lincoln Water System's (LWS's) water supply and identifies alternatives to expand those supplies through the year 2060. This planning effort includes a review of hydrological data to determine the reoccurrence interval of prolonged drought events, an evaluation of the current well performance data to determine the seasonal firm capacity of the well field, a comparison the firm capacity of the well field to projected future water demands, and an evaluation of alternatives to add sufficient supply to account for any projected water deficit.

2.0 Source Water Availability

A hydrologic analysis was performed to evaluate streamflow conditions in the Platte River. The long-term yield of the City of Lincoln's (City's) raw water supply is correlated to the streamflow in the Platte River; therefore, understanding the flow regime of the river is an important part of the Master Plan effort. The objective of this analysis is to determine reoccurrence interval (or frequency) of prolonged droughts and to understand the duration of these events.

2.1 Hydrologic Analysis

Low-flow or drought analysis can be performed using precipitation, soil moisture, or streamflow data. The choice of method depends on the intended use of the water. The Platte River drains more than 85,000 square miles, and for LWS, whose intended use requires knowledge of streamflow, the most appropriate method is to use streamflow as a measurement of drought severity.

A suite of low-flow analysis tools was used to study and quantify drought severity, duration, and frequency. This includes studying the available gage data, creating flow duration curves and daily flow percentiles, calculating annual minimums, calculating the drought frequency in a similar manner to calculating flood frequency, and performing a threshold analysis. Descriptions of each type of analysis and the subsequent results are presented in the following sections.

2.1.1 Available Gage Data

There are four main U.S. Geological Survey (USGS) operated flow gages on the lower Platte River, one gage each at North Bend, Nebraska; Leshara, Nebraska; Ashland, Nebraska; and Louisville, Nebraska. Table 2-1 lists each gage along with its period of record, drainage area, and mean daily discharge. The gage at Leshara is the closest upstream gage to the City well field, and the gage at Ashland is the closest downstream gage. The next upstream gage, at North Bend, was included in the analysis because it has a much longer period of record than the Leshara gage. For similar reasons, the gage at Louisville, which is located further downstream, was included as well.

Table 2-1 Long-term USGS Gage Stations along the Lower Platte River

USGS Gage Number	Gage Name and Location	Drainage Area (sq. mi.)	Mean Daily Discharge (cfs)	Period of Record	Comments
06796000	Platte River at North Bend	70,400	4,938	1949–2012	Available approved daily discharge from April 1, 1949, to 2012.
06796500	Platte River near Leshara	NA	4,834	1994–2012	Available approved daily discharge data from June 29, 1994, to 2012.
06801000	Platte River near Ashland	84,200	6,543	1988–2012	Available approved daily discharge data from September 1, 1988, to 2012.
06805500	Platte River at Louisville	85,370	8,273	1953–2012	Available approved daily discharge data from June 1, 1953, to 2012.

Notes:
sq. mi.= square miles
cfs = cubic feet per second
NA = not applicable

Figures 2-1 through 2-4 show the period of record hydrographs for all four gage locations. The hydrographs show that the lower Platte River typically has a steady baseflow punctuated by precipitation-driven, high-flow events. Upstream of Ashland, the dominant source of the steady baseflow is the Loup River.

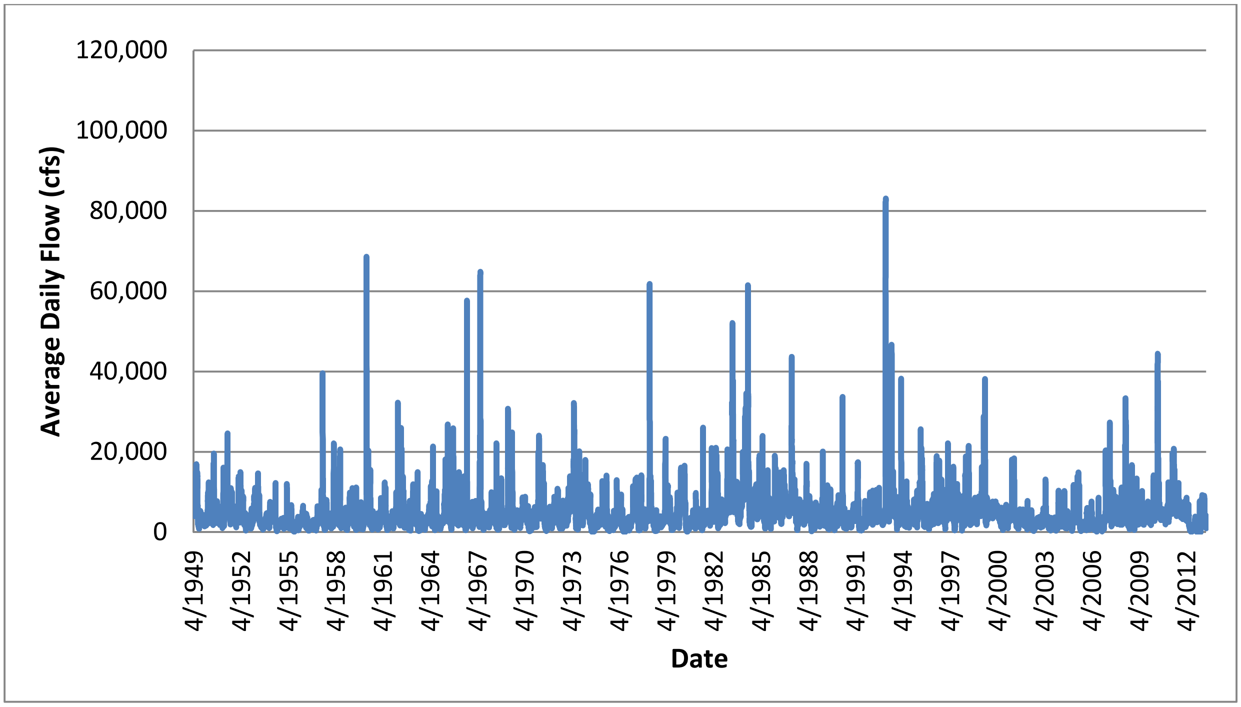


Figure 2-1 USGS Gage at North Bend

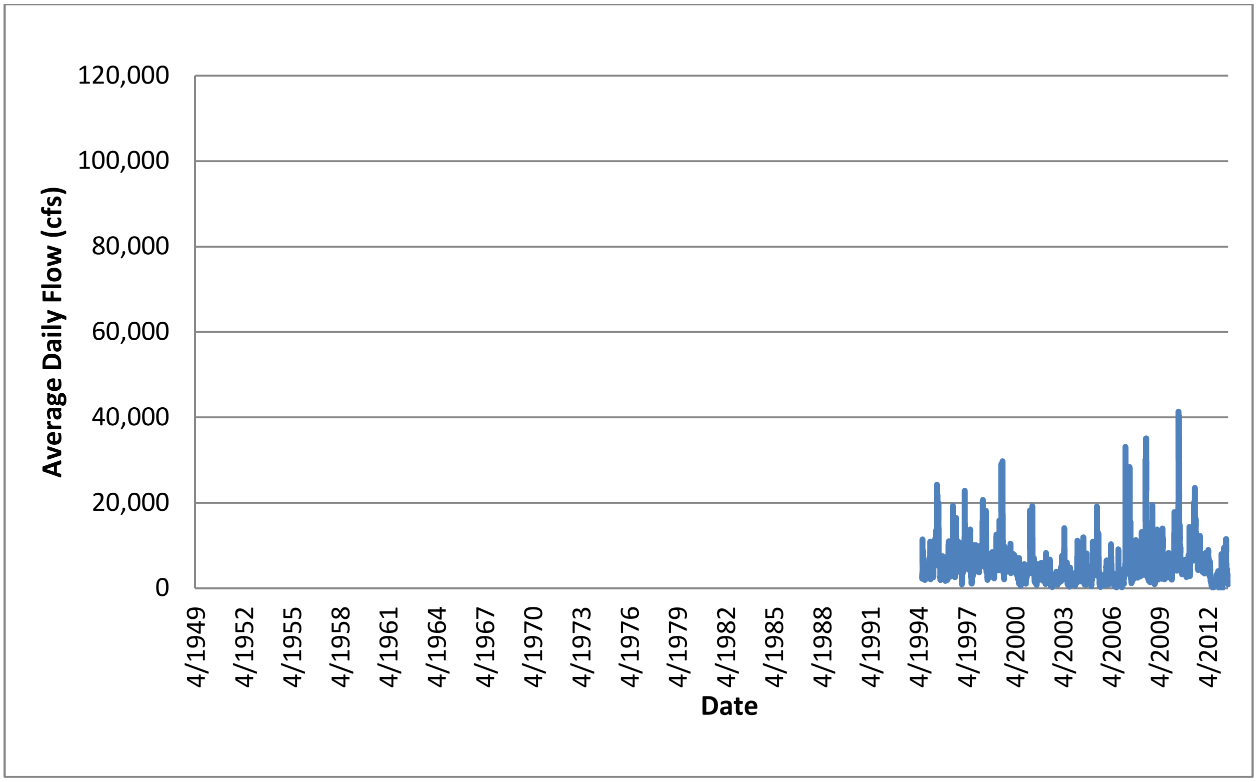


Figure 2-2 USGS Gage near Leshara

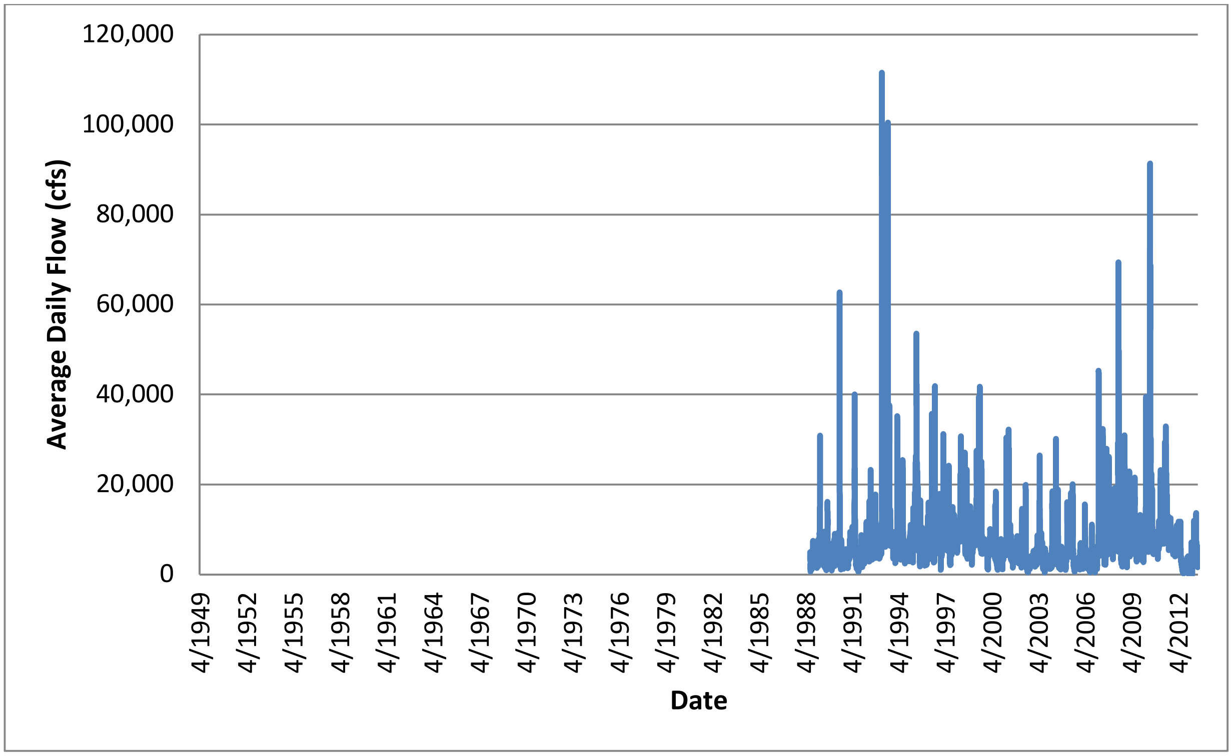


Figure 2-3 USGS Gage near Ashland

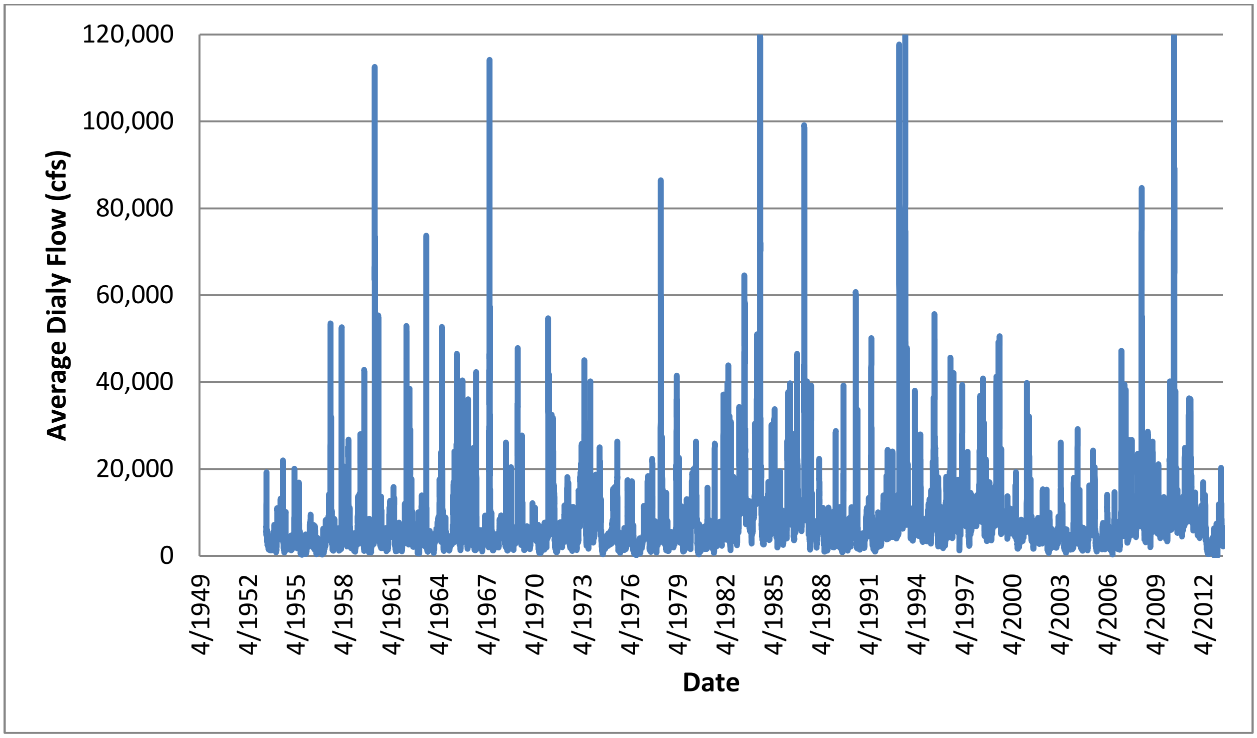


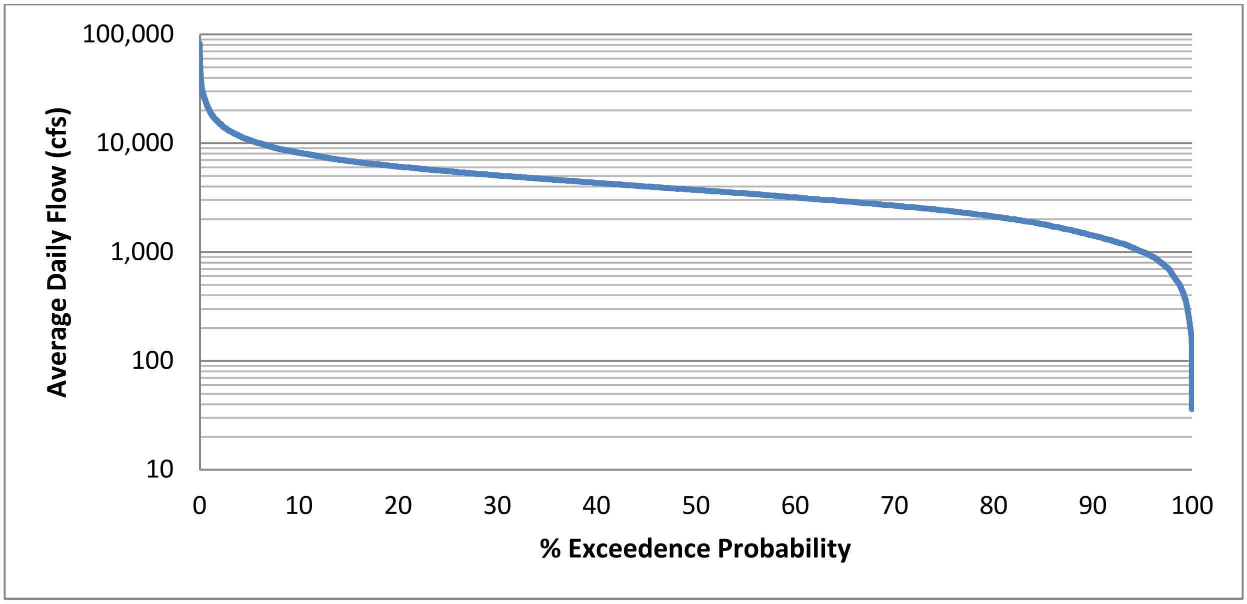
Figure 2-4 USGS Gage at Louisville

2.1.2 Flow Duration

The flow duration curve is a cumulative frequency curve that shows the percentage of time for which specified discharges were equaled or exceeded during a given period. It combines in one curve the flow characteristics of a stream throughout the range of discharge, without regard to the sequence of occurrence.

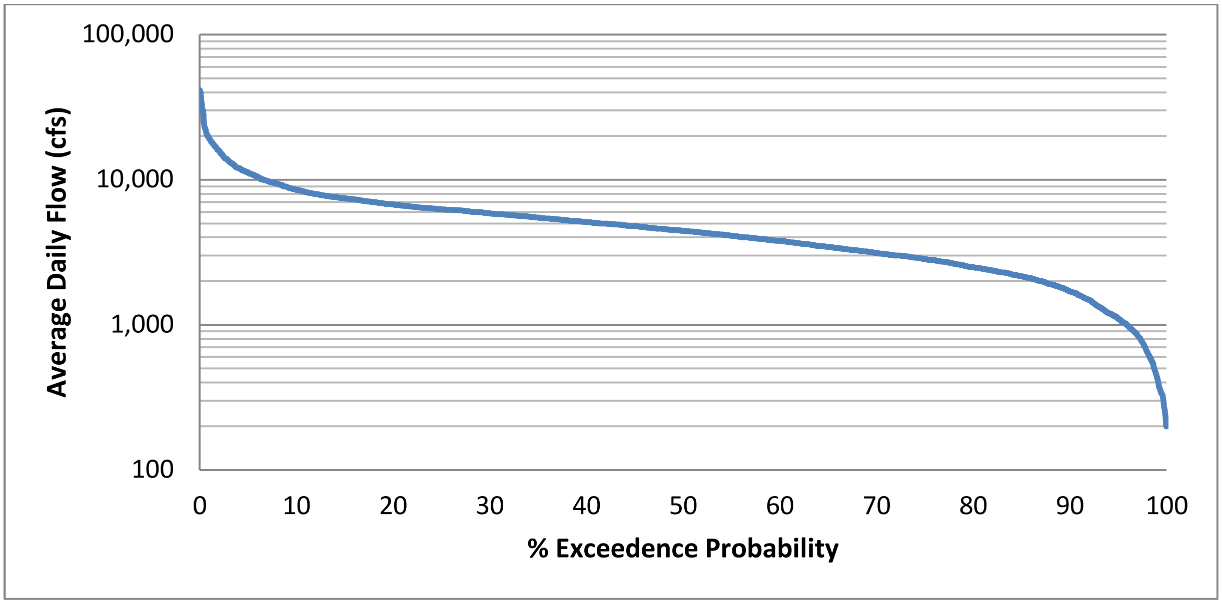
The flow duration curve plots average daily streamflow versus the percent exceedance probability. Percent exceedance is a way to describe the percentage of time for which an observed streamflow is greater than or equal to a defined streamflow. Low-flow events have high exceedance percentages while high-flow events have low exceedance percentages. Low-flow events typically correlate to high exceedance percentages because a majority of the time, actual flows exceed low flow events. The median flow has a 50 percent exceedance probability.

Figures 2-5 through 2-8 show the flow duration curves for the four lower Platte River gages. For all four gages, the 90 percent exceedance is generally less than 2,000 cubic feet per second (cfs). That is, if the flow duration curve represented a typical flow year, the gaged flow would be less than 2,000 cfs for approximately 36 days. The flow duration curves show that the median flows at the four gages range from approximately 3,000 to 7,000 cfs. The median flows are less than the mean flows calculated from the same periods of record. The data shows a skew towards higher flow events which is understandable as very high flows are experienced during flood events, but the lower flows cannot be below zero.



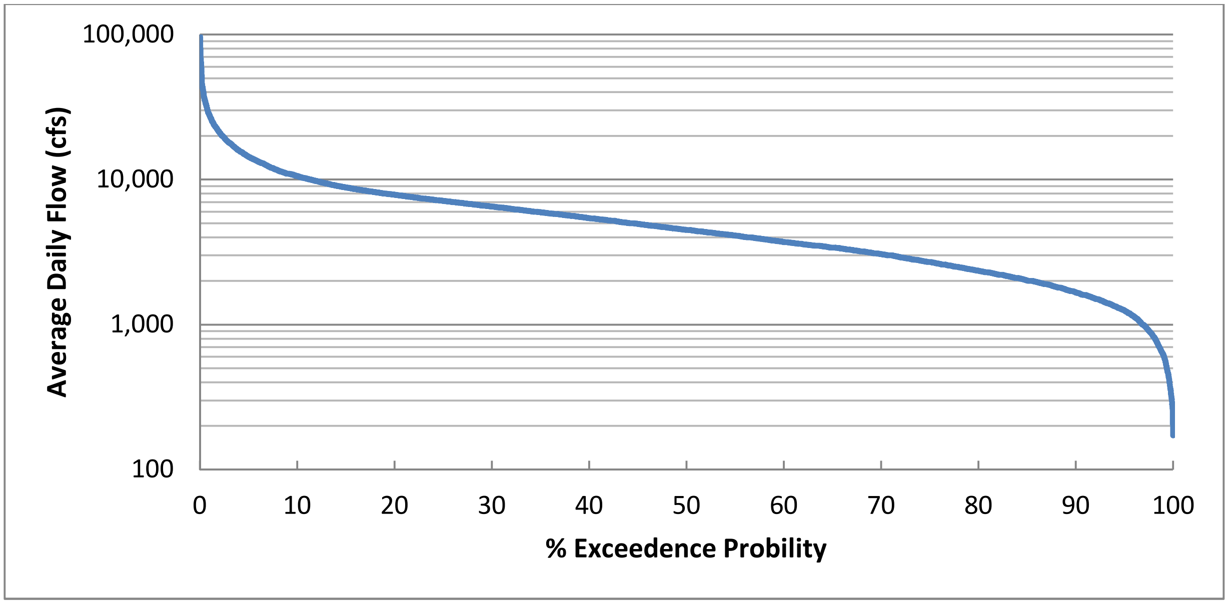
Note:
Curve indicates the percentage of time when observed streamflow is greater than or equal to a defined value.

Figure 2-5 Flow Duration Curve for USGS Gage at North Bend



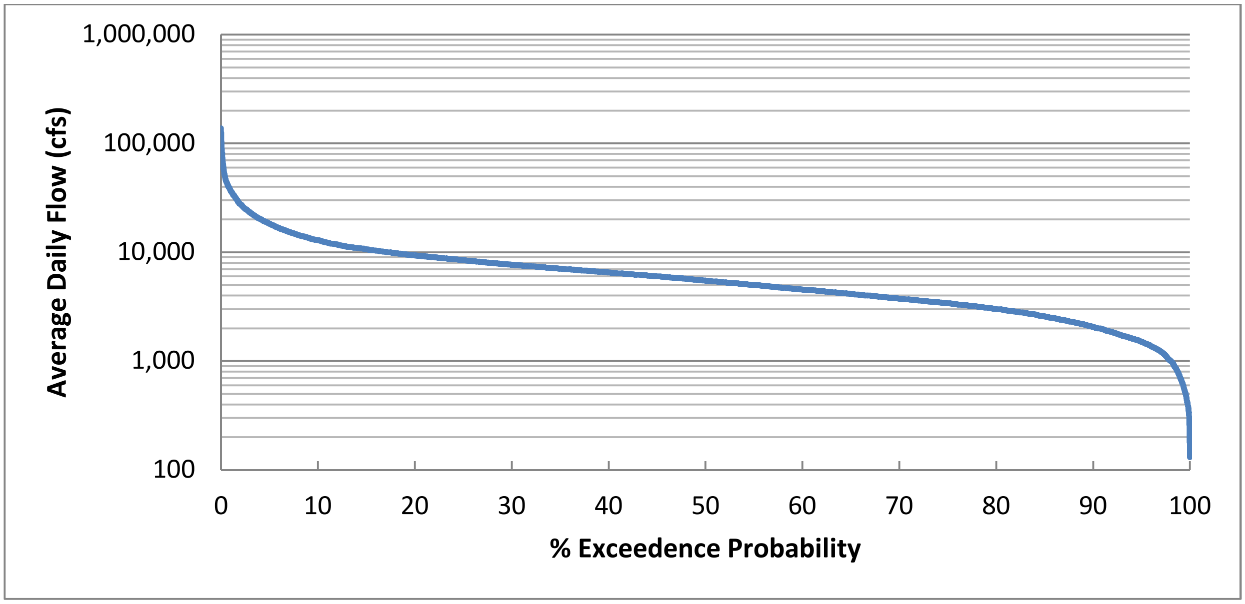
Note:
Curve indicates the percentage of time when observed streamflow is greater than or equal to a defined value.

Figure 2-6 Flow Duration Curve for USGS Gage near Leshara



Note:
Curve indicates the percentage of time when observed streamflow is greater than or equal to a defined value.

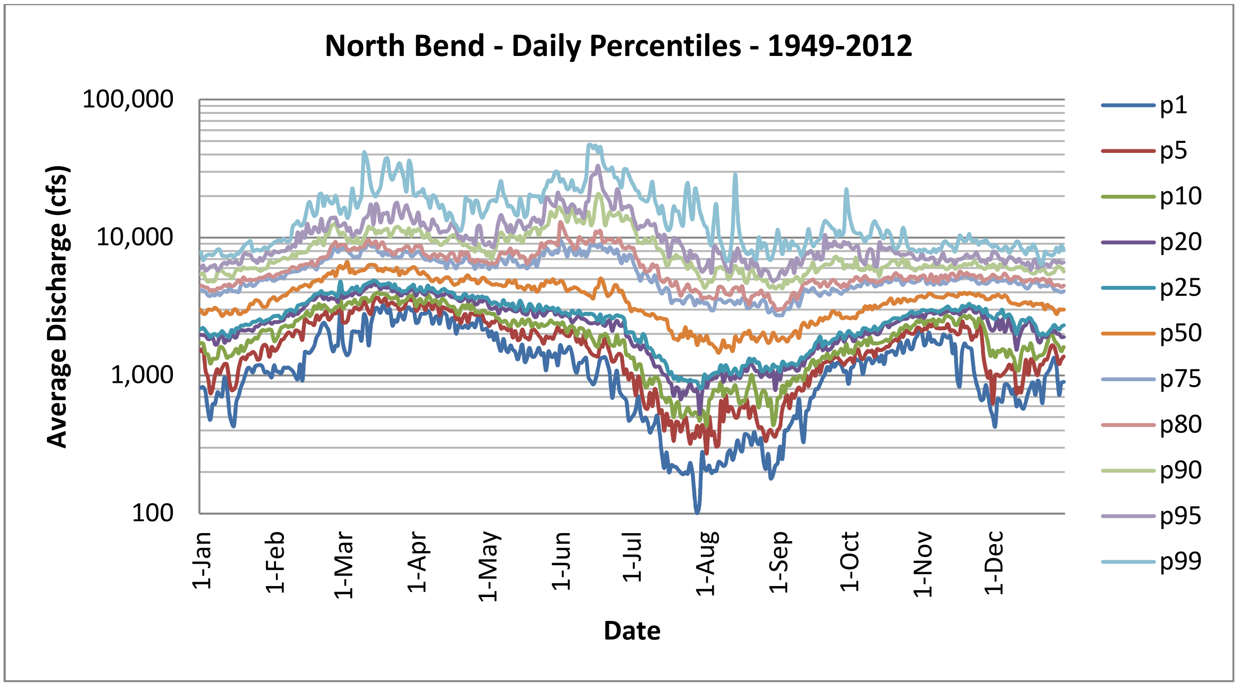
Figure 2-7 Flow Duration Curve for USGS Gage near Ashland



Note:
Curve indicates the percentage of time when observed streamflow is greater than or equal to a defined value.

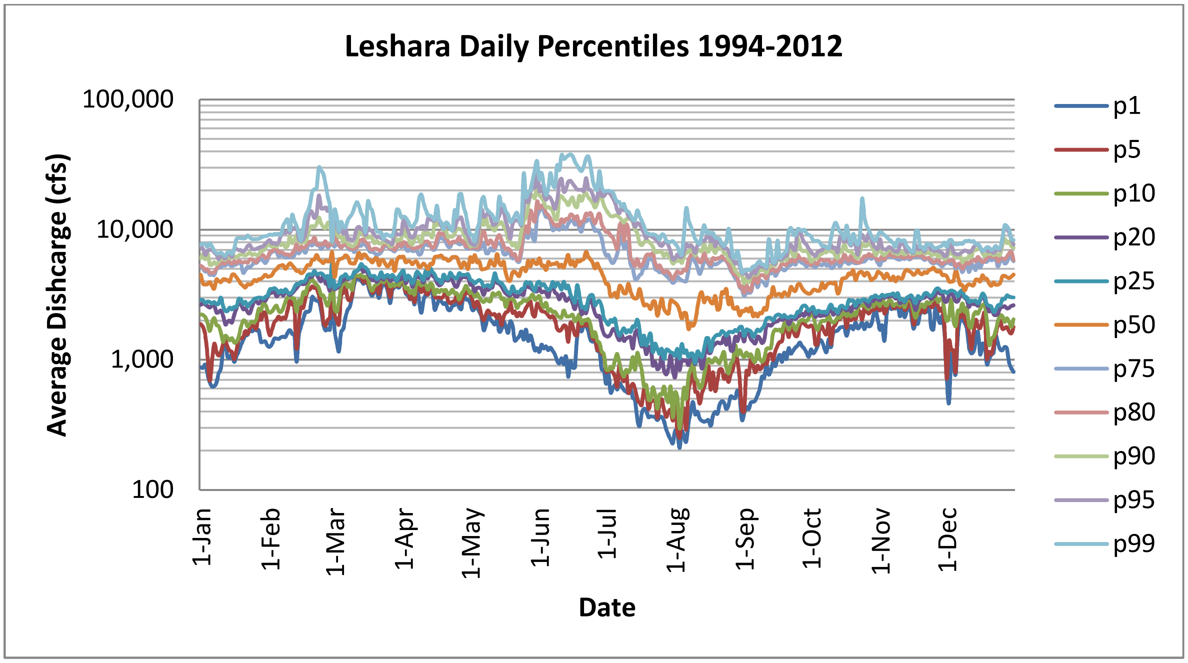
Figure 2-8 Flow Duration Curve for USGS Gage at Louisville

Another way to look at flow frequency is to organize the period of record flow data by each day of the year. For example, all of the January 1 flow data from each year in the period of record is grouped together. This process is repeated for every day of the year. Then flow percentiles are calculated for each day's data. Figures 2-9 through 2-12 show the grouped annual data and each day's associated flow percentiles. The p values referenced in the plots are the percentile values. A percentile is a value on a scale of 1 to 100 that indicates the percentage of a distribution that is equal to or below that value. For example, on the map of daily streamflow conditions, a river discharge at the 75th percentile is equal to or greater than 75 percent of the discharge values recorded on this day of the year during all years that measurements have been taken. The 50th percentile represents the median flow. The plots show that for all four gages, the spring months are typically higher flow times and the summer months experience the lowest flows of the year.



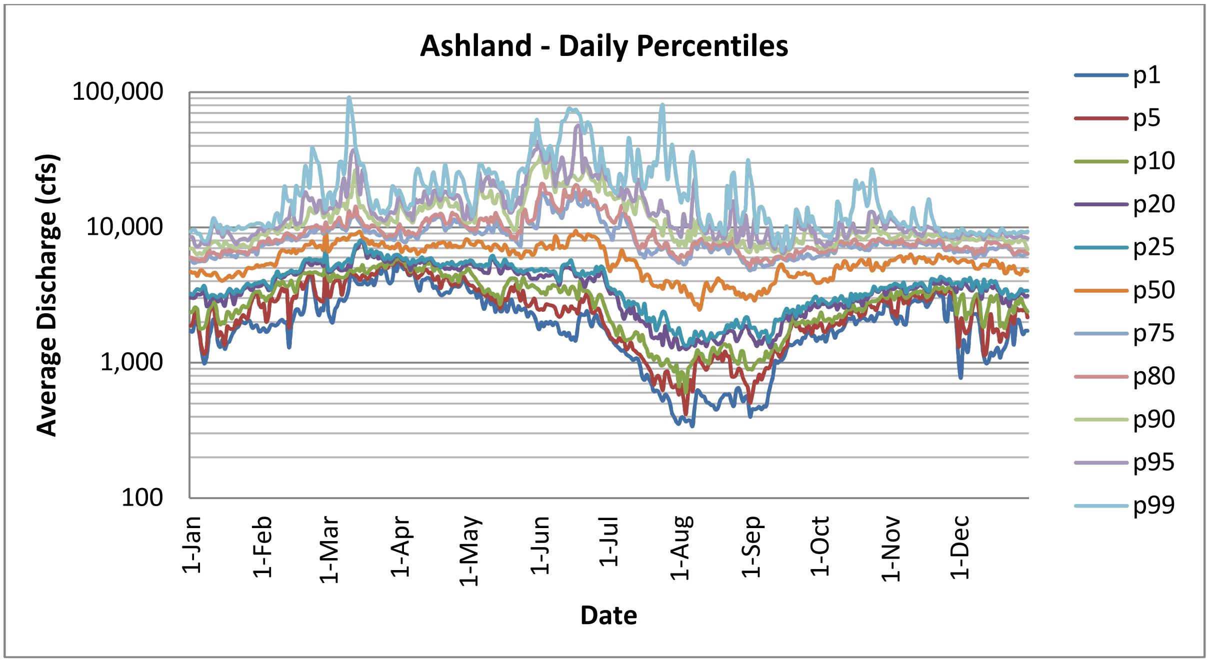
Note:
Percentile lines indicate the streamflow percentage that is equal to or below that value for a given day.

Figure 2-9 Daily Percentiles for USGS Gage at North Bend



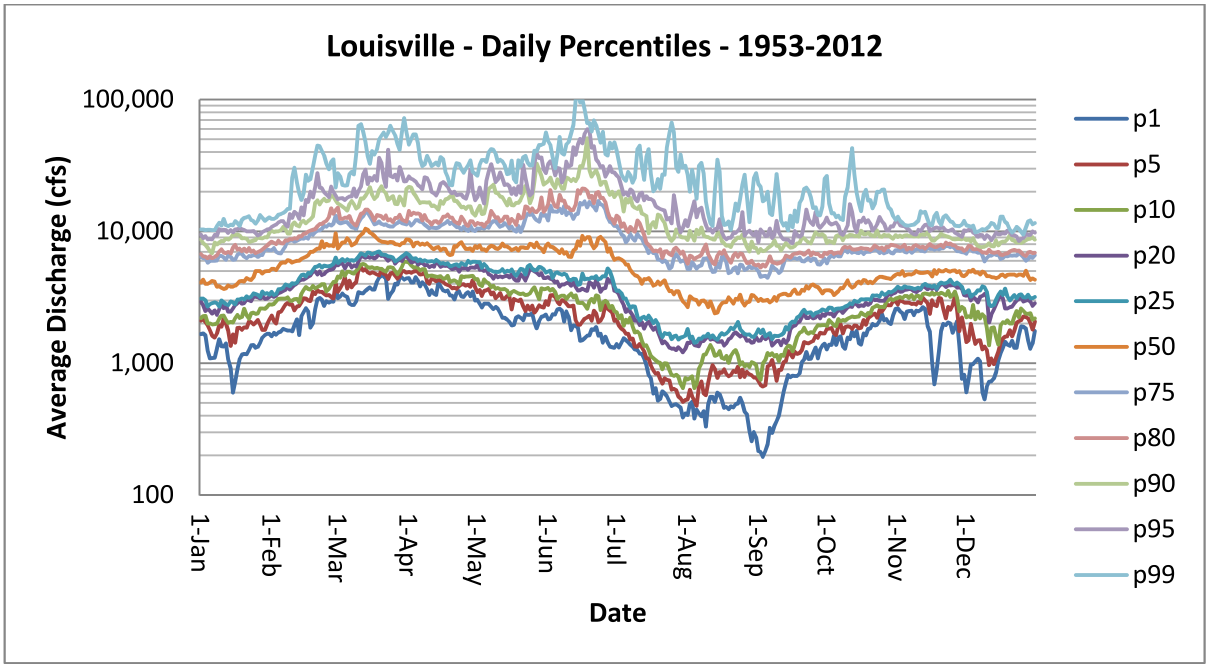
Note:
Percentile lines indicate the streamflow percentage that is equal to or below that value for a given day.

Figure 2-10 Daily Percentiles for USGS Gage near Leshara



Note:
Percentile lines indicate the streamflow percentage that is equal to or below that value for a given day.

Figure 2-11 Daily Percentiles for USGS Gage near Ashland



Note:
Percentile lines indicate the streamflow percentage that is equal to or below that value for a given day.

Figure 2-12 Daily Percentiles for USGS Gage at Louisville

2.1.3 Annual Minimum Flows

Sections 2.1.1 and 2.1.2 showed the different ways flows at the four gages can be described. This and the following sections, Sections 2.1.3 through 2.1.5, describe analyses and procedures for describing and quantifying drought. The drought analysis presented in this section is an annual minimum flow analysis where the 1-day and 30-day annual minimum flows are plotted for the period of record for each gage. Figures 2-13 through 2-20 present the annual minimum flows for each of the four gages being analyzed.

As expected, the 1-day annual low flows are much lower than the 30-day annual low flows. These plots show that using the North Bend and Louisville gages, with their longer periods of record, is important because they clearly demonstrate the difference between hydrologically dry years and wet years. They also show an approximate 10-year cycle of dry years and wet years. This cyclical analysis matches well with the Nebraska Department of Natural Resources' (NDNR's) recommendation of using 25 years of data to capture at least two dry and two wet time periods along the Platte River to calculate basin appropriation status (NDNR 2013). Lastly, these analyses provide a ground trothing to the statistical analyses that are presented in Section 2.1.4.

Statistical frequency analysis is an important tool in hydrologic studies, but it is sometimes beneficial to look at the flood of record, or in this case, the drought of record, using actual data. These data show that at Leshara and Ashland, the year 2012 had the lowest flows on record at both of these stations. The data from North Bend, which extend farther into the past, show that some years in the 1970s had lower flows, but 2012 was among the driest on record. The year 2012 was one of the driest at Louisville as well, but because Louisville receives water from both the Elkhorn River and Salt Creek, Louisville did not experience as low of a flow, comparatively, as the other gages did.

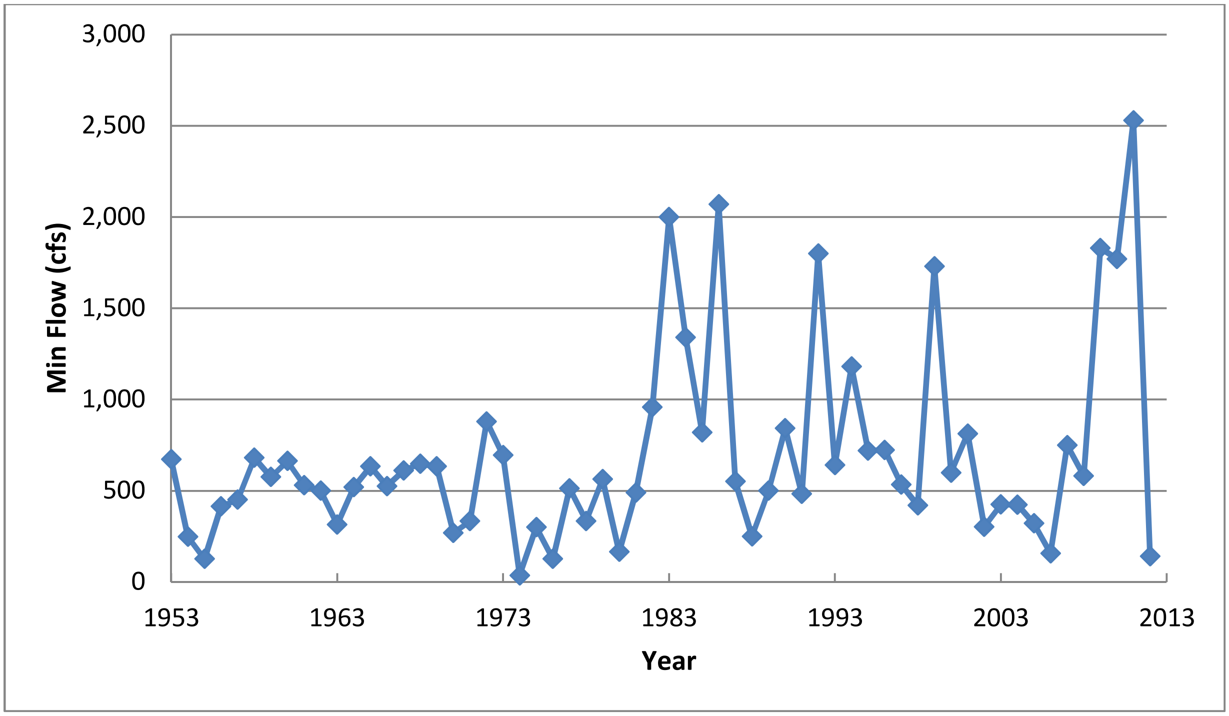


Figure 2-13 1-day Minimum Flow for USGS Gage at North Bend

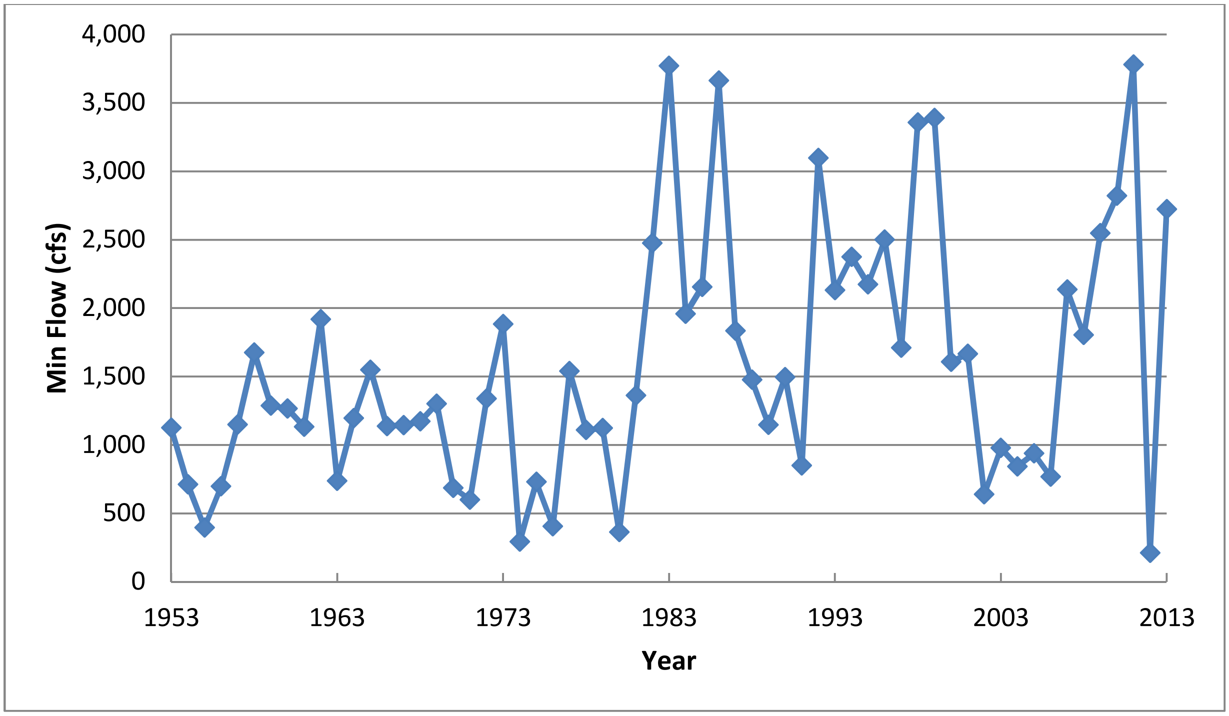


Figure 2-14 30-day Minimum Flow for USGS Gage at North Bend

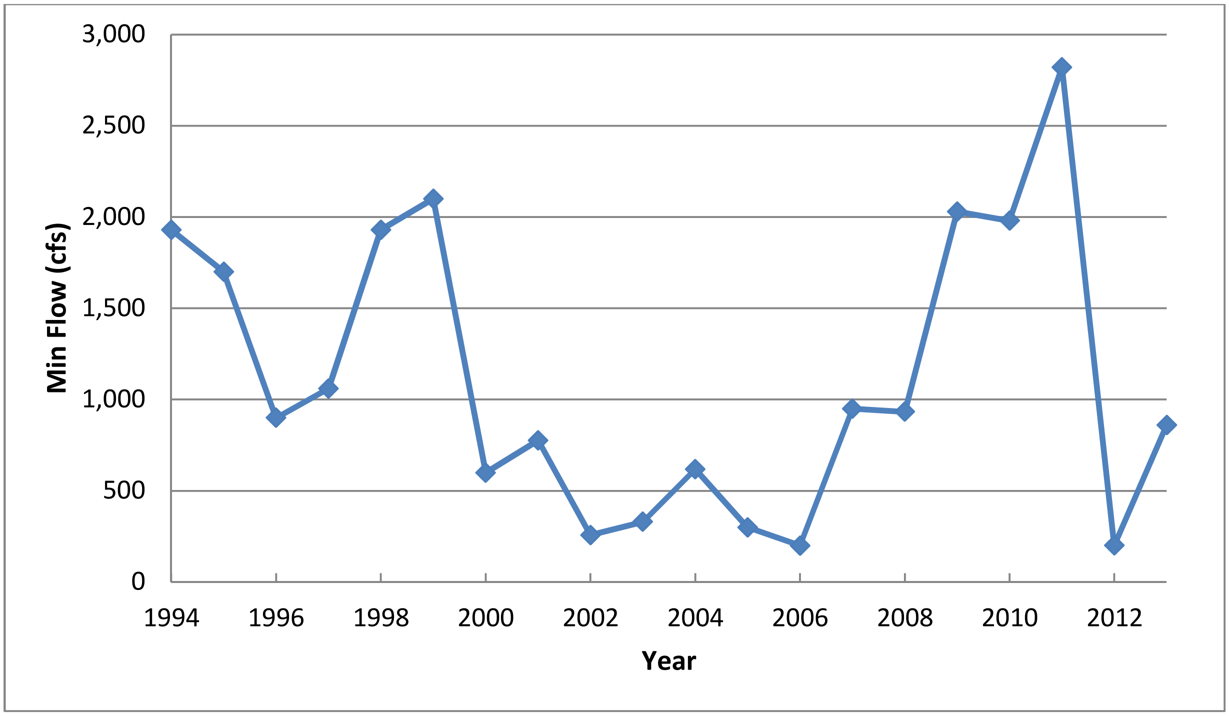


Figure 2-15 1-day Minimum Flow for USGS Gage near Leshara

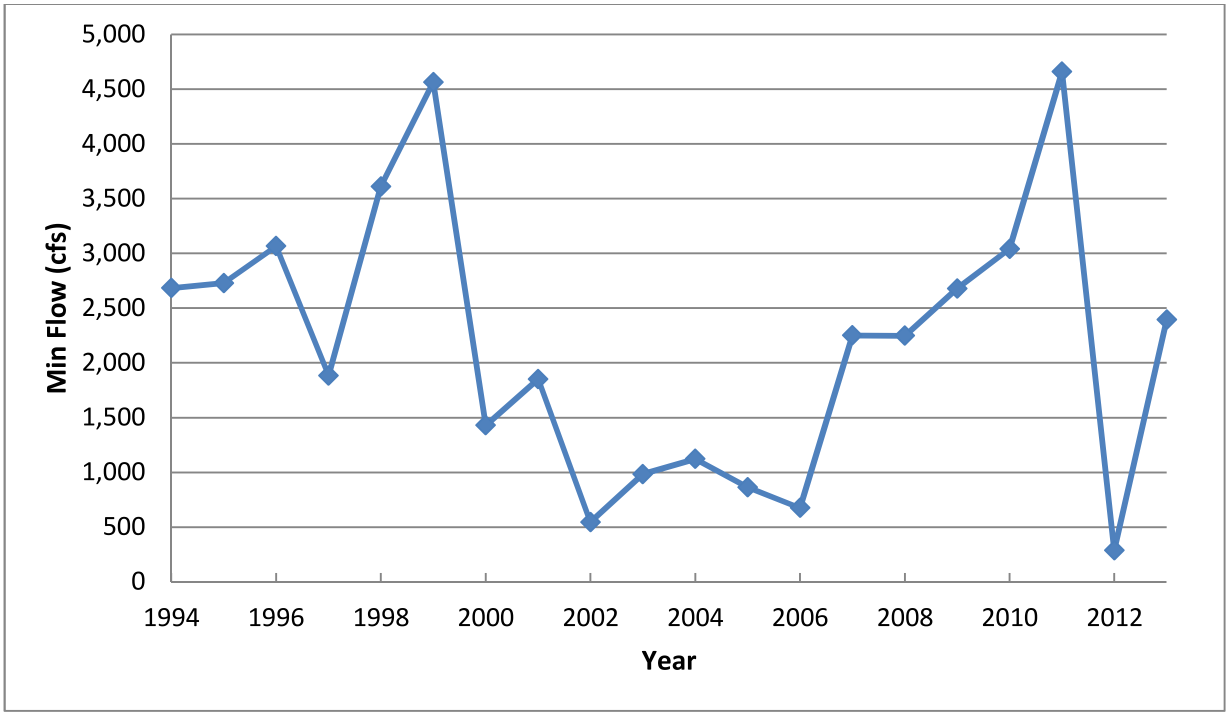


Figure 2-16 30-day Minimum Flow for USGS Gage near Leshara

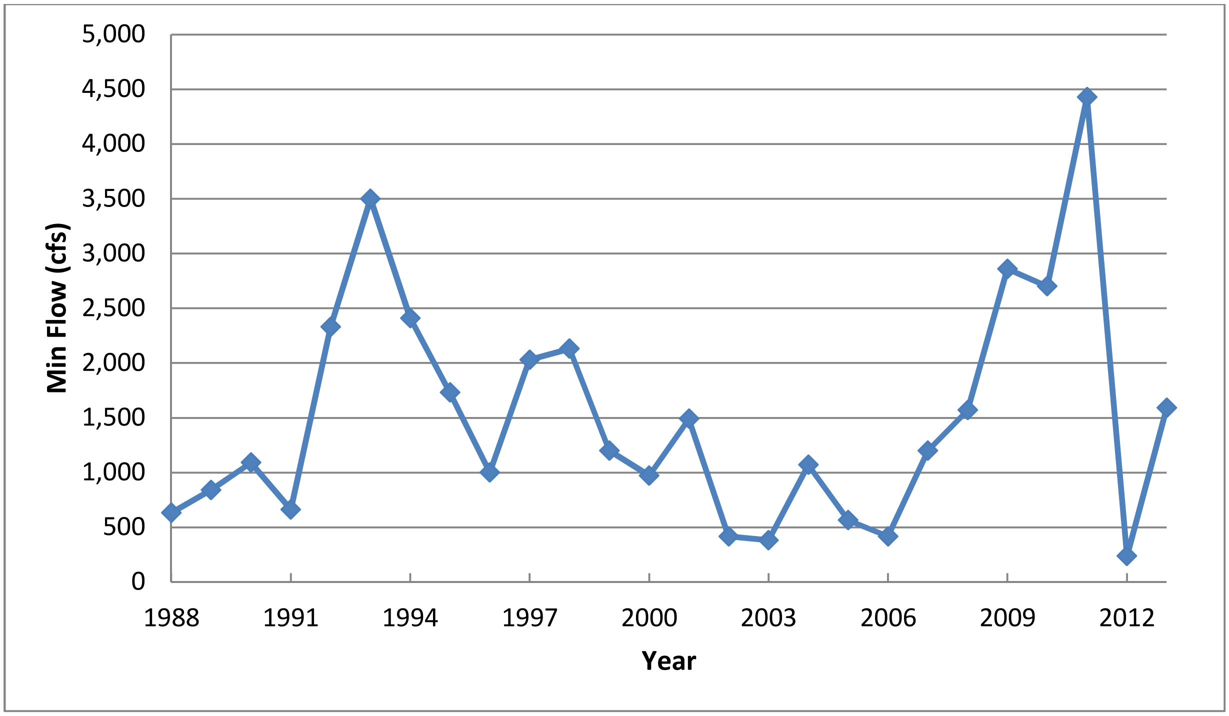


Figure 2-17 1-day Minimum Flow for USGS Gage near Ashland

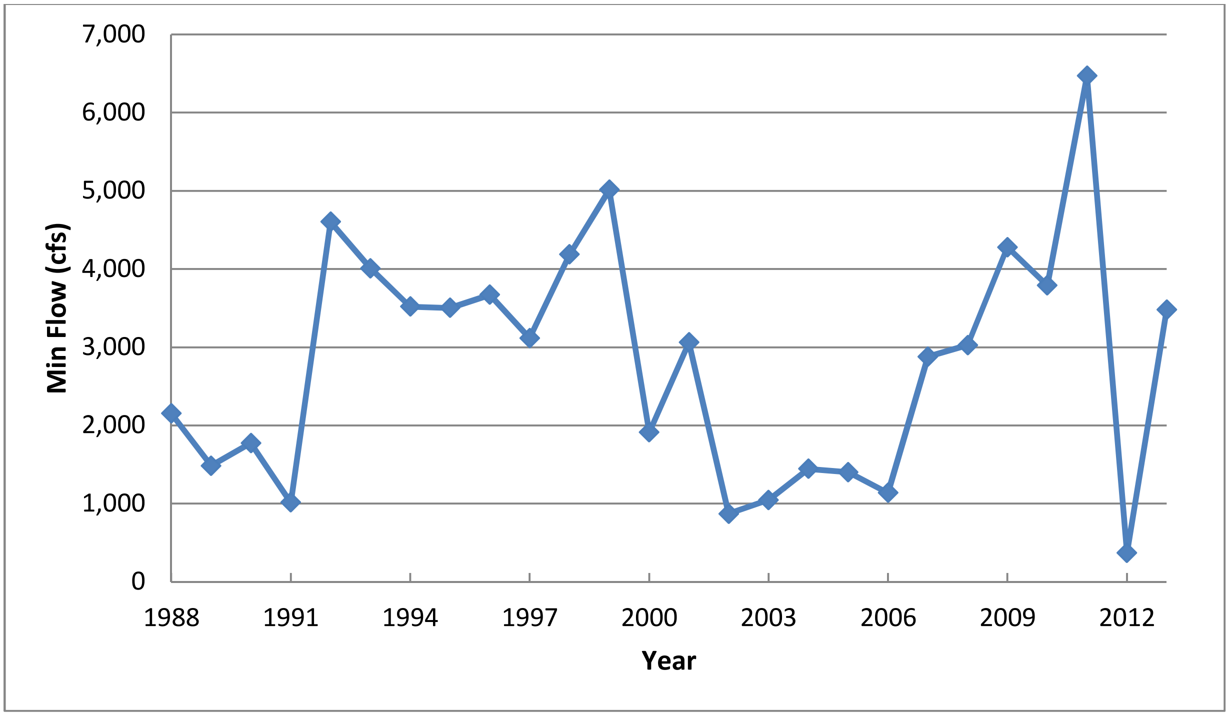


Figure 2-18 30-day Minimum Flow for USGS Gage near Ashland

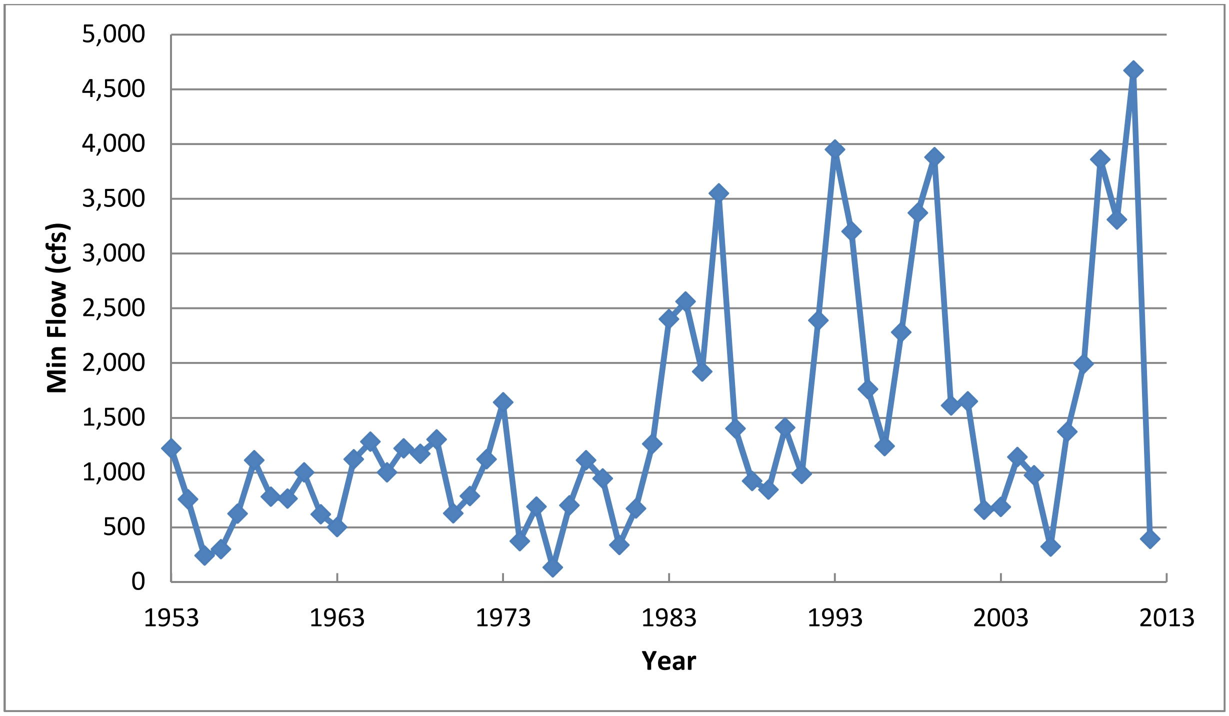


Figure 2-19 1-day Minimum Flow for USGS Gage at Louisville

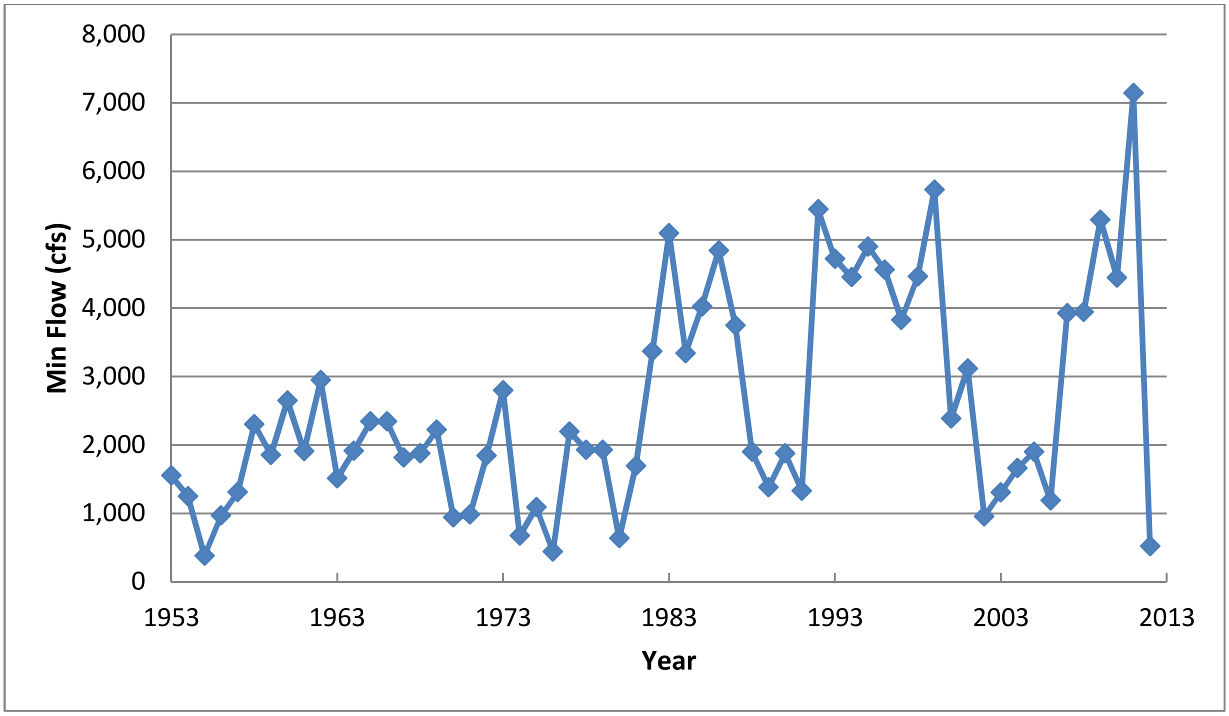


Figure 2-20 30-day Minimum Flow for USGS Gage at Louisville

2.1.4 Drought Frequency Analysis

Flood flow frequency analysis is a common method of ranking flood flows for design purposes. A drought frequency analysis is similar in that both analyses use the same statistical methodologies. The difference is that the drought frequencies quantify the frequency of low flows instead of high flows. Another difference between the flood flow frequency and drought frequency analysis is that droughts are typically given in terms of both duration and frequency. For flood flows, one could calculate the 50-year flow or the 100-year flow. For drought frequency analysis, one could calculate a 3-day, 50-year, 3 day drought flow, or a 10-year, 7-day drought flow.

Drought frequencies and durations were calculated at the four gage locations. The U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center Statistical Software Package, (HEC-SSP) was used for the calculations (USACE 2010). The U.S. Environmental Protection Agency (EPA) flow frequency program, DFLOW, was used as a check to ensure that the HEC-SSP program was supplying reasonable values (EPA 2013). HEC-SSP is an industry standard program for calculating flood flow frequencies. DFLOW is commonly used by National Pollutant Discharge Elimination System (NPDES) permitted facilities because NPDES permits typically require the use of the 10-year, 7-day low flow.

Tables 2-2 through 2-5 show the drought, low-flow frequency statistics for all four gage locations. The return periods calculated are the 1-, 2-, 4-, 5-, 6-, 10-, 12-, 20-, 50-, 100-, 200-, and 500-year return periods. The durations calculated for each low-flow frequency calculation are 1, 3, 7, 15, and 30 days. These durations and frequencies cover a wide range of drought severities. An additional analysis was performed at the Ashland gage, where 60- and 90-day duration droughts were also calculated; these are the durations that are most commonly used for evaluating the available supply for the City well field.

Table 2-2 North Bend Drought Frequency

Frequency		Streamflow (cfs) for Event Duration (days)				
Percent Chance Exceedance	Return Period (years)	1	3	7	15	30
99	1	2,130	2,222	2,600	3,298	3,890
50	2	655	711	842	1,104	1,358
25	4	394	443	536	701	880
20	5	344	391	476	621	785
17	6	308	353	432	564	715
10	10	234	274	339	441	566
8	12	212	251	312	405	522
5	20	165	200	252	325	424
2	50	109	136	176	225	299
1	100	81	104	137	174	233
0.5	200	60	80	107	135	184
0.2	500	42	58	79	99	136

Table 2-3 Leshara Drought Frequency

Frequency		Streamflow (cfs) for Event Duration (days)				
Percent Chance Exceedance	Return Period (years)	1	3	7	15	30
99	1	5,029	4,833	5,227	6,075	6,605
50	2	894	960	1,140	1,413	1,673
25	4	506	568	684	823	975
20	5	438	497	600	716	846
17	6	391	448	543	640	754
10	10	295	347	421	484	567
8	12	269	318	387	440	514
5	20	211	255	311	345	399
2	50	143	179	218	230	261
1	100	110	141	171	174	194
0.5	200	86	112	136	133	146
0.2	500	63	85	103	95	103

Table 2-4 Ashland Drought Frequency

Frequency		Streamflow (cfs) for Event Duration (days)						
Percent Chance Exceedance	Return Period (years)	1	3	7	15	30	60	90
99	1	5,574	5,377	5,570	6,662	7,205	8,839	10,003
50	2	1,278	1,442	1,700	1,987	2,301	2,624	2,989
25	4	782	894	1,053	1,216	1,416	1,597	1,869
20	5	690	789	928	1,068	1,244	1,401	1,653
17	6	626	716	838	962	1,119	1,259	1,497
10	10	491	560	649	740	860	965	1,170
8	12	452	515	594	675	784	880	1,074
5	20	367	415	473	534	617	693	863
2	50	262	292	323	360	413	466	600
1	100	207	228	247	273	311	351	465
0.5	200	167	181	191	210	237	269	365
0.2	500	128	135	138	150	168	191	269

Table 2-5 Louisville Drought Frequency

Frequency		Streamflow (cfs) for Event Duration (days)				
Percent Chance Exceedance	Return Period (years)	1	3	7	15	30
99	1	5,082	5,099	5,384	6,418	7,659
50	2	1,179	1,279	1,484	1,799	2,086
25	4	709	775	902	1,095	1,310
20	5	623	680	792	961	1,161
17	6	562	614	713	865	1,056
10	10	435	475	549	666	833
8	12	399	435	502	608	768
5	20	320	347	398	482	625
2	50	223	240	271	327	445
1	100	174	186	207	250	352
0.5	200	137	146	161	193	282
0.2	500	103	108	116	139	214

Based on this analysis, the 2012 drought can be characterized as presented in Table 2-6. The variation in drought frequency amongst the four gages is due to the different periods of record for the gages and the differing contributions from tributaries to the Platte River. Depending on the location and duration, the drought experienced in 2012 ranged from a 12-year event up to nearly a 200-year event.

Table 2-6 2012 Drought Frequency for Various Durations

Station	Period of Record	Duration (days)		
		7	30	60
North Bend	1949–2012	50 Year	100 Year	200 Year
Leshara	1994–2012	20 to 50 Year	20 to 50 Year	20 to 50 Year
Ashland	1988–2012	50 to 100 Year	50 to 100 Year	50 Year
Louisville	1953–2012	12 to 20 Year	20 to 50 Year	20 to 50 Year

2.1.5 Flow Threshold Analysis

The final drought analysis conducted is called the flow threshold analysis. It is a simple, quantitative way to define the beginning, ending, and severity of a drought. By selecting a threshold flow value, typically obtained from a flow duration curve or a low-flow frequency analysis like those described in previous sections, the number of times the flow drops below the a threshold flow value can be counted. Additionally, as the flow drops below the threshold, the length of time the flow is below the threshold can be observed, as can the minimum flow and average flow during the duration the measured flow was below the selected threshold flow. This method is an easy way to both visualize and quantify the frequency, duration, and severity of observed droughts; the results can then be compared back to the drought frequency analysis and used for planning purposes.

For the purposes of this Master Plan, the 50-year 60-day low flow at each gage was chosen as the threshold value to present an example of the type of information that can be obtained from this type of analysis. This value was chosen as a possible river discharge at which the water supply will be evaluated. Table 2-7 summarizes the threshold frequency information for the Ashland gage, which Figure 2-21 shows, graphically. At the Ashland gage, the measured streamflow was below the 50-year, 60-day low flow (that is, 466 cfs) for a period of nearly 30 days during the summer of 2012.

Table 2-7 Ashland 10-year, 30-day Drought Threshold, 2002-2012

Duration (Days)	Average Flow During Deficit (cfs)	Average Deficit (cfs)	Month	Year
1	416	50	8	2002
1	381	85	9	2003
1	417	49	8	2006
1	428	38	12	2006
29	364	102	7	2012
1	457	9	8	2012
8	392	74	9	2012

Note:
The 50-year, 60-day low flow at Ashland equals 466 cfs.

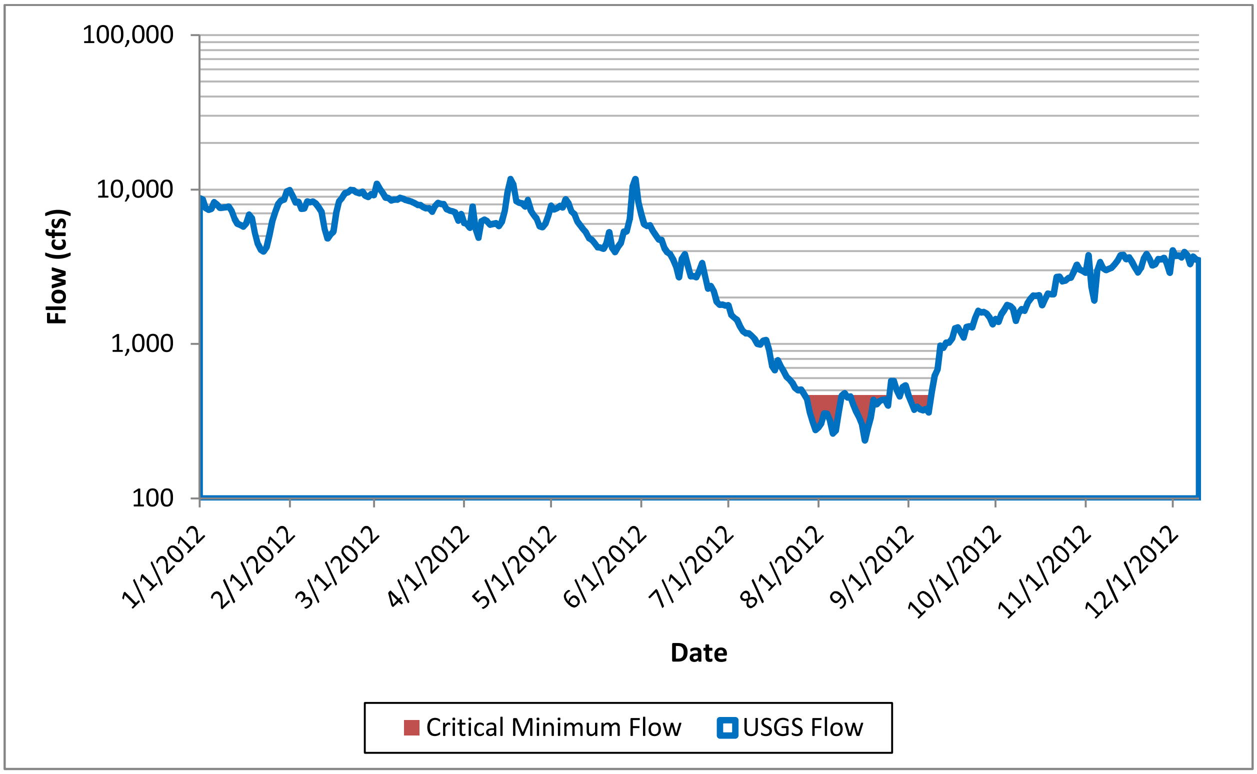


Figure 2-21 Ashland 10-year, 30-day Drought Threshold, 2012

2.2 Summary of Hydrologic Analysis

The drought experienced by the City during the summer of 2012 was an event that was highly influenced by extreme low-flow conditions on the Platte River that were observed upstream of North Bend. The data from the North Bend gage show that there were periods in the 1970s with lower streamflow, but the 2012 flows were among the lowest on record. Specifically, at North Bend, the 2012 drought was a 100- to 200-year reoccurrence interval drought for the 30-day and 60-day streamflow periods, respectively. The 2012 drought was one of worst on record at Louisville, as well. However, because Louisville receives water from both the Elkhorn River and Salt Creek, Louisville did not experience as low of a flow, comparatively, as was observed at North Bend.

Based on the analyses performed, the drought experienced by the City (at the Ashland gage location) during the summer of 2012 was a 50- to 100-year reoccurrence interval event for the 7-day and 30-day low-flow duration events, and a 50-year reoccurrence interval event for the 60-day duration event. The statistical analysis performed indicates that because the planning horizon for this Master Plan is 50 years and the reoccurrence interval of the 2012 event is approximately 50 years, there is a strong probability that the City will experience at least one drought event of similar magnitude during the planning horizon of this Master Plan.

Table 2-8 provides a summary of the 12-, 20-, 50-, and 100-year reoccurrence interval low-flow events for the Ashland gage site, along with the probability that those events could occur during a 50-year planning period. Given the results of this analysis, it is recommended that the availability of the water supply be evaluated over a range of streamflow values that is bracketed by the 50–year, 60-day (466 cfs) and 100–year, 30-day (311 cfs) drought events. To put these planning values into context, the measured minimum 30-day low-flow during the 2012 drought was 368 cfs, and the measured minimum 60-day low-flow was 473 cfs. Previous Master Plan values used 200 cfs as the planning level drought condition.

Table 2-8 Ashland Drought Frequency

Frequency			Streamflow (cfs) for Event Duration		
Probability of Occurring in Any Given Year	Probability of at Least One Exceedance During 50-year Planning horizon	Return Period (years)	30 days	60 days	90 days
8%	99%	12	784	880	1,074
5%	92%	20	617	693	863
2%	64%	50	413	466	600
1%	39%	100	311	351	465

3.0 Supply Analysis

The raw water supply source for the City is a well field located near Ashland. The water available for municipal use is restricted by water rights. The following section evaluates the existing water rights and well field infrastructure to determine the raw water production capacity of the City's well field.

3.1 Water Rights Review

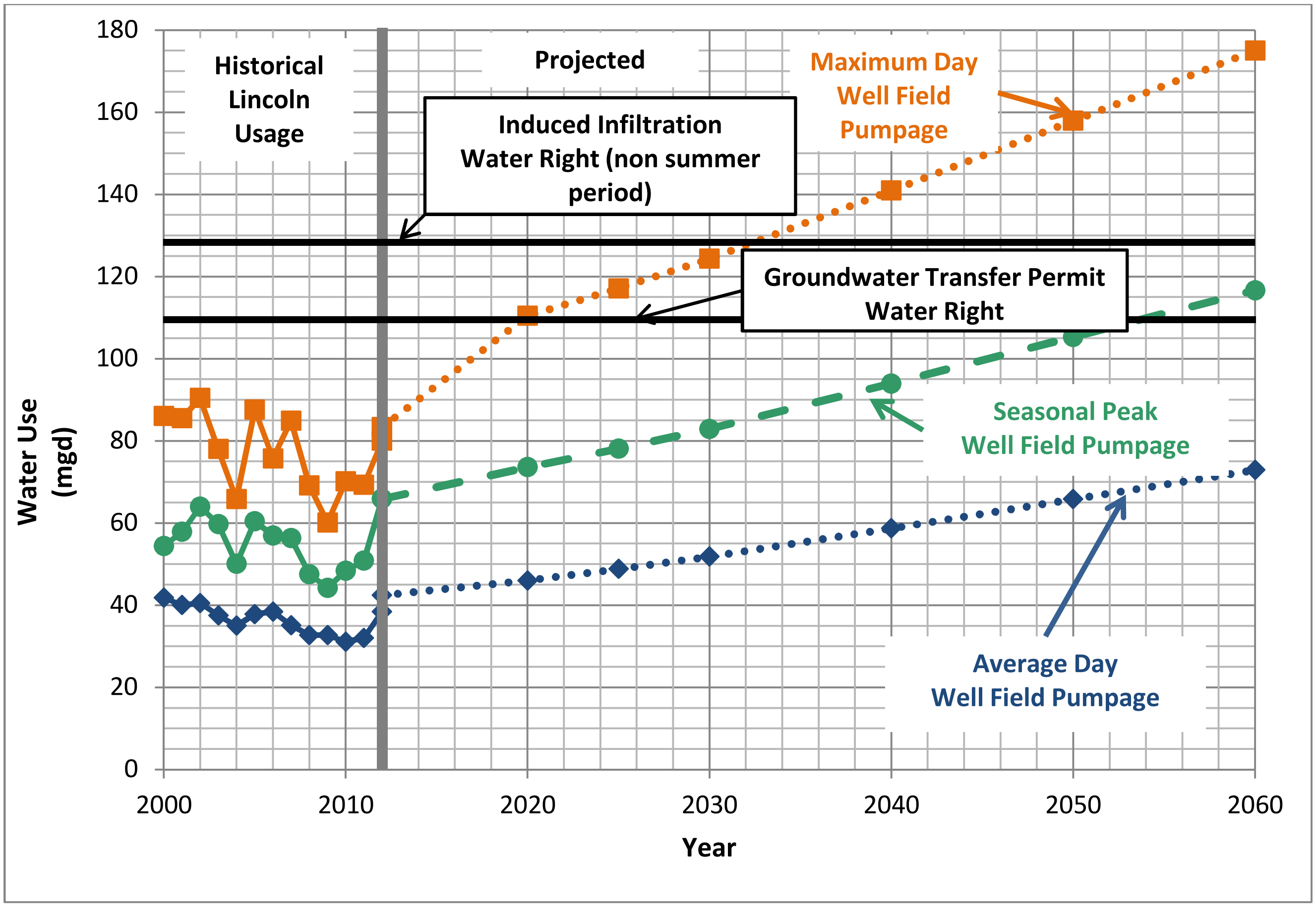
A factor that could potentially limit the raw water availability for LWS is the City's water rights for municipal use. The City maintains two types of water rights permits through NDNR: an induced infiltration permit and groundwater transfer permits. The induced infiltration permit allows the City to induce groundwater recharge from the Platte River for municipal use. The induced infiltration permit is Certificate A-17312, which has the following conditions:

- Water can be diverted from 48 wells, located in the City's well field.
- The stream reach where the City can divert.

- Permitted streamflow amount is limited to:
  - o 704 cfs (454 million gallons per day [mgd]) summer season (May 15 to September 15)
  - o 200 cfs (129 mgd) for the remainder of the year
  - o In the City's Water Management Plan, it is stated that if the Platte River streamflow at the City's well field is below 700 cfs for 5 continuous days, the City will consider asking the State of Nebraska to administer its water rights.
- A variable priority date. Priority date of 1964 for 31 wells. The other wells have priority dates ranging from 1970 to 1993.
- A requirement that new or replacement wells must be located and constructed to take reasonable advantage of aquifer conditions in the area. Reasonable advantage is defined in a Settlement Agreement between the City and the Central Platte Natural Resources District, dated September 14, 1998. That document defines reasonable advantage as a condition in the aquifer where the drawdown measured near the wells is equal to a minimum of 25 percent of the original aquifer thickness.

The groundwater transfer permits maintained by the City are Certificates A-10367 and A-16917. These Certificates allow the City to withdraw groundwater from one location and transport and use it elsewhere. Certificate A-10367 allows the City to produce up to 60 mgd, and Certificate A-16917 allows the City to produce up to 50 mgd. Certificate A-16917 limits the total number of wells in the well field to 40 vertical wells and 4 horizontal collector wells (HCWs). Therefore, the total limitation of groundwater that can be produced and transferred by the City is 110 mgd. The City also maintains a groundwater transfer permit to divert 12 mgd from the Antelope Valley wells. These wells are no longer operated because of water quality concerns.

The available water rights were compared to the water demand projections presented in *Chapter 2 - Water Capacity Requirements* to determine if the existing water rights are sufficient to meet the projected water demands over the planning horizon of this study. As shown below in Figure 3-1, the 200 cfs (129 mgd) induced infiltration limitation exceeds the average day water raw water production rate projected for 2060 (that is, 72 mgd). Therefore, the available water rights are sufficient for the projected average day water use for the entire planning horizon. The 704 cfs (454 mgd) limitation placed on water diversions from May 15 through September 15 exceeds the maximum day water production rate projected for 2060 (that is, 166 mgd), indicating that the induced infiltration right will not limit the raw water availability needed to meet maximum day demand. The 110 mgd groundwater transfer permit limitation is exceeded by the maximum day demand in 2022 and by the seasonal peaking factor in 2054. Therefore, it is recommended that the City begin discussions with NDNR to modify the existing groundwater transfer permit or to obtain a new groundwater transfer permit.



Note:

1. *The projected Maximum Day Pumping values from 2020 to 2060 were reduced by 5 mgd from the values presented in Chapter 2, Water Capacity Requirements, to account for water conservation measures.*

Figure 3-1 Water Rights and Projected Demand

3.2 Well Field Pumping Capacity

The City's well field is located near Ashland. The well field consists of 40 vertical wells, 2 existing HCWs, and an additional HCW currently under construction. When construction on third HCW is complete, the City's well field will have a total pumping capacity of 192 mgd and a total set pumping capacity of approximately 149 mgd. The caisson for a fourth HCW is under construction but the remaining components of the well would need to be constructed in the future for the HCW to be operational.

3.3 Well Condition Assessment

3.3.1 Vertical Well Condition Assessment

The 2002 Facilities Master Plan (2002 Master Plan) presented a methodology to identify which wells should be rehabilitated or abandoned and re-drilled. That methodology established the specific capacity of the well as the criteria for making these determinations. Generally, the specific capacity of a well gradually decreases over time and will temporarily increase after well rehabilitation. A decline in specific capacity indicates that the well is unable to produce the same flow rate at a given drawdown and can be an indication of mechanical well screen plugging from sediment or fouling of the well screen from mineral incrustation or biological activity. The following procedures were recommended in the 2002 Master Plan for scheduling well maintenance activities:

- When the specific capacity of a well falls between 40 and 50 gpm/ft of drawdown, the well should be rehabilitated.
- If the well does not respond to rehabilitation and the specific capacity of the well falls below 40 gpm/ft of drawdown, the well should be abandoned and replaced.

This methodology was one of the criteria used to determine the condition of the wells for the Master Plan. In addition, the following guidelines were used for recommending wells for rehabilitation:

- Compare the current specific capacity to the percentage of original specific capacity remaining in the well. In *Groundwater and Wells* (Driscoll 1995), Driscoll recommends treating wells after the specific capacity has declined to 75 percent of the original specific capacity. This value is widely used in practice to schedule well maintenance.
- Evaluate the specific capacity trend plots of each well, which were provided by LWS. If a sharp downward trend was noted, the well was recommended for rehabilitation.

The decline in the performance of a well cannot be predicted, and neither can the increase in specific capacity after rehabilitation. Therefore, it is not possible to predict which wells or how many wells will have to be rehabilitated or replaced over the planning horizon of this Master

Plan. However, the wells that are currently operating below the performance metrics identified are summarized below. These wells should be considered for well rehabilitation:

- Seven wells currently have a specific capacity below 50 gpm/ft.
- Four wells currently exhibit a sharp downward trend in specific capacity and could potentially benefit the most from an aggressive well rehabilitation or a more detailed evaluation.
- Twenty six wells are operating at less than 75 percent of their original specific capacity.

Based on the analysis performed, the following priority order is recommended for well rehabilitation or replacement:

1. Wells that have a sharp downward trend in specific capacity and have a specific capacity below 50 gpm/ft.
2. Wells that have a sharp downward trend in specific capacity or have a specific capacity below 50 gpm/ft.
3. Wells identified that are operating at less than 75 percent of their original specific capacity.

If any well that exhibits a sharp downward trend in specific capacity does not respond to a standard well treatment, it is recommended that a detailed evaluation of that well be performed by a licensed well driller or pump installer. This evaluation should provide a recommendation for a more aggressive well rehabilitation or for replacement of the well.

3.3.2 Horizontal Collector Well Condition Assessment

A review of the performance of the two existing HCWs is performed annually by Ranney Collector Wells (Layne 2013). This annual monitoring program was initiated in 2003. The annual evaluations are typically limited to a review of monitoring data; however, in 2010, a physical inspection of the HCWs was performed. The inspection included a structural inspection of the caisson and flow tests that were performed by isolating each lateral.

The HCWs continue to maintain a high enough specific capacity and there is a sufficient amount of available drawdown above the lateral screens that both wells should continue to produce at or above their design yield of 17.5 mgd for the foreseeable future (Layne 2013). Redevelopment of either of these wells is not recommended at this time. However, like vertical wells, the decline in the performance of a HCW cannot be predicted, and it is not possible to predict when these wells may need to be rehabilitated. Therefore, it is recommended that the City continue the annual monitoring program for these wells and that the data collected from this program be used to make decisions regarding maintenance of the HCWs.

3.4 Well Field Transmission System Analysis

This section presents information concerning the existing well field water transmission system and a hydraulic model update, and results of a hydraulic analysis performed evaluating the transmission system under existing and future maximum day supply conditions.

3.4.1 Existing Transmission System

The raw water transmission mains carry water from the City's well field to the Platte River Water Treatment Facility. There is approximately 104,400 feet (19.8 miles) of existing transmission mains ranging from 8 to 54 inches in diameter. Main materials include cast iron, ductile iron, pre-stressed concrete, and reinforced concrete.

3.4.2 Well Field Transmission Main Hydraulic Model Update

Updates to the geographic information system (GIS) transmission main diameters were completed to match provided system schematic information. Along with other well facility data, the updated GIS transmission mains were used as the basis of the model update.

The initial model received from LWS was converted from H_2OMAP to InfoWater (Version 10.0 Update 7) by Innovyze. This enabled the raw water transmission system model to be accessed within the Esri GIS environment and to use the same software package and version as the potable water transmission and distribution systems model.

The infrastructure and facilities in the model were updated based on revised GIS transmission mains and latest available facility information. The well pump curves and reservoir information, representing the groundwater supplies, were checked in the model and updated where necessary using the 2011 pumping test results. Drawdown over time was not accounted for in the model at the wells because these analyses were focused on the transmission system flow. If the well pump performance is to be evaluated closer in the future, drawdown should be accounted for. The transmission main connections into the water treatment plants were updated to more closely match GIS data and aerial information.

The model pipe Hazen-Williams roughness factors (C values) in the previous model were used and range from 80 to 130 depending on pipe material and age. The future pipe improvements added to the model use a C value of 130, which is a typical C value for newer pipe materials such as polyvinyl chloride or ductile iron. The valve head loss coefficients were also used from the previous model.

The primary purpose of the model update and well field transmission system analysis is to determine if the system is adequately sized for existing and 2025 raw water supply flows. Two steady-state scenarios were developed for the raw water transmission system analysis: a base year (2012) model validation scenario and a 2025 (short-term) planning period scenario. These scenarios represent maximum day well field supply flows as determined in *Chapter 2 - Water Capacity Requirements*.

3.4.2.1 Model Validation

Two sources of flow data were used to validate the model: individual well flows and the major transmission mains into the Platte River Water Treatment Facility. The maximum day of raw water supply on July 21, 2012, was chosen as the flow scenario for model validation. This day represents the greatest flows supplied by the wells to the water treatment plants to simulate the maximum flows in the transmission system for evaluation of velocity, head loss, and pressures. The total validation flows in the model of 82.8 mgd are less than 1 percent different than the historical flow on July 21, 2012, of 83.3 mgd.

3.4.2.2 Future Transmission System Improvements

The proposed well field improvements, consisting of two HCWs (third and fourth HCW) and associated piping, were added to the model. The third HCW is planned to be constructed by the middle of 2014, and the fourth HCW is recommended to be operational by 2018 as discussed in Section 4.0. Therefore, both future wells and their piping were added to the Year 2025 (short-term) scenario.

An estimated flow of 15 to 20 mgd is expected from each new HCW based on the pump design flow rates. Only two pumps in each proposed HCW were activated in the model, and they were run at reduced speeds (90 to 95 percent) to help balance flows between the four HCWs. The raw water supply flows from the two proposed HCWs in addition to the existing vertical wells and HCWs should supply enough water to meet the Year 2025 raw water supply demands.

3.4.3 Model Results and Analysis

The transmission system was analyzed under raw water supply flow conditions for the base year (2012) and 2025 (short-term) maximum day well field pumpage projections developed in *Chapter 2 - Water Capacity Requirements*. Pipe velocity and pressures from the model results were evaluated to determine if any pipelines were reaching their capacity under the estimated supply flow conditions as described in more detail in the following sections. Normal pipe velocities are 0 to 7 feet per second (fps) before becoming excessive for this type of system. Normal pressure ranges between 10 to 100 pounds per square inch (psi) for this system depending on the location in the system.

3.4.3.1 Base Year Conditions

The base year (2012) conditions include a total of 82.8 mgd being delivered by the raw water transmission system to the water treatment plants. Approximately 56.0 mgd are fed to the West Plant and 26.8 mgd are fed to the East Plant. The modeling results indicate that there are neither pressures nor velocities out of the normal expected range for each area of the system.

3.4.3.2 2025 Conditions

The 2025 (short-term) conditions include a total of 117 mgd being delivered by the raw water transmission system to the water treatment plants. The modeling results indicate that there are neither pressures nor velocities out of the normal expected range for each area of the system. The velocities in the transmission mains feeding the East Plant increased substantially with the addition of the two proposed HCWs. The velocities are within the range of 5 to 7 fps, but do not exceed 7 fps. Based on this, it appears that the proposed pipeline sizes are adequate to handle the additional flow from the two HCWs.

3.4.3.3 Transmission System Capacity

The raw water transmission system capacities were established by evaluating the total flow in the pipelines that would increase pipeline velocities above 7 fps and/or impose head losses in the system that would push well pumps back on their curves by more than 15 percent outside of their most efficient operating range. This was accomplished in the model by adding future well supply flows near the ends of the transmission mains and observing the changes in velocities and the existing well pump operation points.

3.4.4 Transmission System Recommendations

This section presents the recommended improvements or operational changes to the raw water transmission system to provide additional water supply to the water treatment plants and operate the system more efficiently.

3.4.4.1 Base Year Conditions

There are no recommended capital improvements to the well field transmission system in the short-term by 2025.

3.4.4.2 2025 Conditions

The third HCW is planned to be online by mid-2014. The fourth HCW is recommended to be online by 2018 to meet near-term raw water supply requirements. With the two proposed HCWs added to the raw water transmission system with capacities of 15 to 20 mgd each, the existing and proposed pipelines would be able to handle the flows to provide the 2025 raw water supply requirement to the treatment plants. Therefore, no additional pipeline is recommended by 2025.

3.4.4.3 Transmission System Capacity Evaluation

Based on the transmission system capacity evaluation, the total transmission system capacity is estimated at approximately 145 mgd.

By 2025, the transmission main capacity will be reached with the addition of the fifth HCW, as discussed in Section 4.0, and only if the HCWs are running at an average flow of 15 mgd each. By 2035, when the sixth recommended HCW is built, an additional transmission main will be required to convey all HCW water to the East Plant.

Depending on the location of any additional future wells in the Platte River Water Treatment Facility raw water system, additional hydraulic analysis should be completed to determine the specific impact on existing pipeline and well performance as well as to establish the most optimal use of spare capacity in the raw water transmission system.

3.5 Well Field Firm Capacity

The firm capacity of the well field is defined as the total production capacity with the largest unit out of service. For the City's well field, the largest production unit is a pump in one of the HCWs. The firm capacity of the well field is governed by two separate criteria: 1) the aquifer, which must be capable of yielding the volume of water needed; and 2) the hydraulic capacity of the wells, pumps, and pipelines. Because of the seasonal variability in aquifer conditions, the firm capacity of the well field is also seasonal. This section evaluates the aquifer yield and the resulting well field firm capacity of the existing well field.

3.5.1 Methodology

There are two different methods that can be used to estimate the aquifer yield and firm capacity of the well field. The first method is to use a groundwater flow model, which can be used to predict the changing aquifer conditions that occur within a well field during a time of high demand. These conditions include quantifying the amount of well interference drawdown between wells, quantifying the amount of streamflow infiltration induced by pumping, and calculating the time varying change in aquifer thickness. Although groundwater models are powerful tools for evaluating these local scale changes in aquifer conditions, they are typically not refined enough to accurately predict drawdown within an individual pumping well, and they do not account for well inefficiency, which also increases drawdown in the well. Drawdown within the pumping well is typically the limiting factor on the production rate of that well and, therefore, a significant factor in estimation of the firm capacity of a well field.

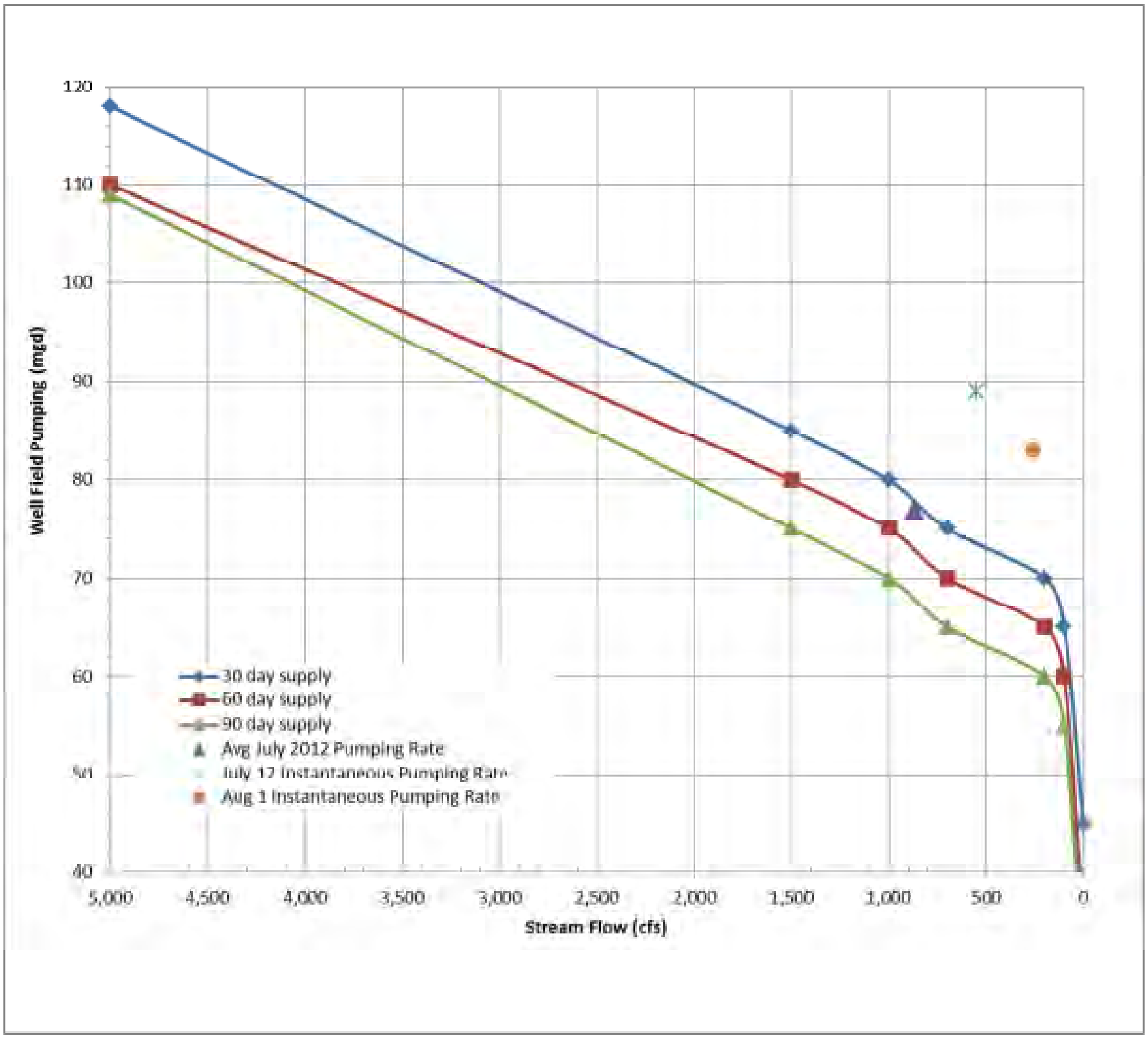
The second method for estimating the firm capacity of a well field is to use the results of well capacity tests that are conducted within the well field. During these tests, accurate measurements of flow and drawdown within the pumping well are collected, allowing for the determination of the specific capacity of the well. This allows the hydrogeologist to accurately predict the maximum pumping rate of the well, which is typically limited by the drawdown in the pumping well. However, unlike using a model, using in-well tests does not allow for the calculation of well interference drawdown (that is, drawdown induced from one pumping well and observed in another pumping well), nor does this method allow for the calculation of the time varying decrease in aquifer thickness that occurs during prolonged periods of pumping.

Therefore, the most reliable estimate of the firm capacity of a well field should include both the use of a groundwater flow model and an analysis of the well capacity tests. The following section describes the analysis used to estimate the firm capacity of the City's well field.

3.5.2 Well Field Seasonal Yield – Groundwater Modeling

The 2002 Master Plan evaluated the aquifer yield using a numerical model of the well field that was developed with USGS groundwater modeling code MODFLOW. LWS has contracted with a consultant to update this model. HDR Engineering, Inc. (HDR) reviewed the documentation provided on these modeling efforts and compared the predictions from the model to the actual well field pumping that occurred during the summer drought of 2012.

The groundwater model was developed using several assumptions, the most conservative being to limit drawdown in any one cell to 25 percent of the available drawdown. The results of various modeling simulations are presented below in Figure 3-2. This figure shows that the model-predicted maximum well field pumping varies based on streamflow and also decreases with increased pumping duration. These model predictions were developed using the well field configuration as of 2003 and do not include a third HCW. The model results summarized in Figure 3-2 were originally presented in the *Drought Response 2002 Technical Memorandum* (Black and Veatch 2002) and were updated in 2012.



Notes:
1. *2012 well field pumping data are shown as data points (not lines).*
2. *Curves indicate the maximum pumping rate that is sustainable for a specific duration of pumping.*
3. *Model results summarized in Figure 3-7 are a graphical representation of the well field summer capacity provided by TZA.*

Figure 3-2 Model-Predicted Well Field Capacity and 2012 Pumping Conditions

In addition to the results of previous groundwater modeling, Figure 3-2 shows the observed streamflow (at the Ashland gage) and pumping conditions during the drought of 2012. As shown, the instantaneous (or daily flow rates) were higher than the model predictions. However, the monthly average well field pumpage of 77 mgd during July 2012, when average streamflow was recorded as 870 cfs, matches the model prediction reasonably well. Based on this reasonable match between the model predictions and the observed performance of the well field, it was concluded that the model appears to be a good tool for evaluating the performance of the well field during periods of drought.

The model results indicate that there is a nearly linear relationship between the well field yield and change in streamflow over a large range of streamflow values. Specifically, the slope of each of the 30-, 60-, and 90-day well field supply curves indicates that for streamflow conditions above 1,000 cfs, there is an approximate drop of 10 mgd in well field flow for each drop of 1,000 cfs in streamflow. The relationship between streamflow and well field yield changes slightly when streamflow is between 1,000 to 150 cfs. At these lower streamflow conditions, for each drop in streamflow of approximately 80 cfs, there is a corresponding decline in the well field yield of 1 mgd. When streamflow is below 150 cfs, the relationship between streamflow and well field yield changes dramatically, which indicates that based on the model results, 150 cfs is a critical streamflow value for the City's well field. At this streamflow condition, it appears that the source of water to the well field changes from predominantly induced infiltration of the Platte River to predominantly groundwater in aquifer storage. A daily streamflow of 150 cfs has never been observed at the Ashland gage. The lowest daily streamflow observed at the Ashland gage is 237 cfs, and the lowest monthly average streamflow observed at this gage is 368 cfs. Both of these record low streamflow values were observed in August 2012.

3.5.3 Firm Capacity – Existing Well Field

Table 3-1 summarizes the analysis of the well field firm capacity developed using the approach described above. The firm capacity values presented in Table 3-1 were developed as follows:

- Maximum instantaneous pumping – Developed using the September 2012 well testing results.
- Maximum pumping for 2 and 3 months – Developed through interpretation of the groundwater modeling results. The maximum well field pumping rates are interpreted from the data presented in Figure 3-2 at the 2 and 3 month streamflow values that correlated to a 100-year reoccurrence drought at the Ashland gage.

The firm capacity values presented in Table 3-1 do not include the additional raw water capacity that will be provided from the third HCW that is under construction.

Table 3-1 City's Well Field Seasonal Firm Capacity

Season	Streamflow (cfs)	Water Level Conditions	Maximum Instantaneous Pumping (mgd)	Maximum Pumping for 2 Months (mgd)	Maximum Pumping for 3 Months (mgd)
Spring	3,000 or greater	High	110	100	95
Summer low flow	1,000 to 500	Low	100	70	65
Summer drought	< 500 (100 Year Reoccurrence Interval Drought)	Low	90	67	61

Note:
The 100–year, 60-day average streamflow during drought at Ashland is 351 cfs. The 100–year, 90-day average streamflow during drought at Ashland is 465 cfs.

3.5.4 Firm Capacity with the third Horizontal Collector Well

Given the groundwater model's reasonable match to actual drought conditions observed during the summer of 2012, it appears that the groundwater model is a reliable tool for estimating the long-term well field yield during a drought. Therefore, the chart presented as Figure 3-2, above, was updated to include the raw water supply from the third HCW in order to estimate the firm capacity of the well field with the third HCW. Table 3-2 summarizes the analysis of the well field firm capacity. The firm capacity values presented in Table 3-2 were developed as follows:

- Maximum instantaneous pumping – Developed using the September 2012 well testing results. Based on a 2013 design memorandum (Black and Veatch 2013), it was assumed that the third HCW would increase the instantaneous pumping capacity by 20 mgd.
- Maximum pumping for 2 and 3 months – 10 mgd of water capacity was added to the maximum well field pumping rates presented in Table 3-1 to account for the increase in well field yield from the new HCW. The 10 mgd value was based on a 2013 design memorandum (Black and Veatch 2013) and is a conservative assumption given the information presented in the 2013 design memorandum.

The updated well field capacity is summarized below in Table 3-2 and Figure 3-3. This figure also includes the drought planning horizon for a 20- to 100-year reoccurrence interval drought at the Ashland gage location for both a 30- and 60-day duration event. In addition, Figure 3-3 includes the observed flow conditions from the 2012 drought, illustrating both the 30- and 60-day low-flow values for the Ashland gage. The seasonal well field capacity with the third HCW is summarized in Table 3-2.

Table 3-2 City's Well Field Seasonal Firm Capacity with the Third HCW

Season	Streamflow (cfs)	Water Level Conditions	Maximum Instantaneous Pumping (mgd)	Maximum Pumping for 2 Months (mgd)	Maximum Pumping for 3 Months (mgd)
Spring	3,000 or greater	High	130	110	105
Summer low flow	1,000 to 500	Low	120	80	75
Summer drought	<500 (100 Year Reoccurrence Interval Drought)	Low	110	77	71

Note:
The 100–year, 60-day average streamflow during drought at Ashland is 351 cfs. The 100-year, 90-day average streamflow during drought at Ashland is 465 cfs.

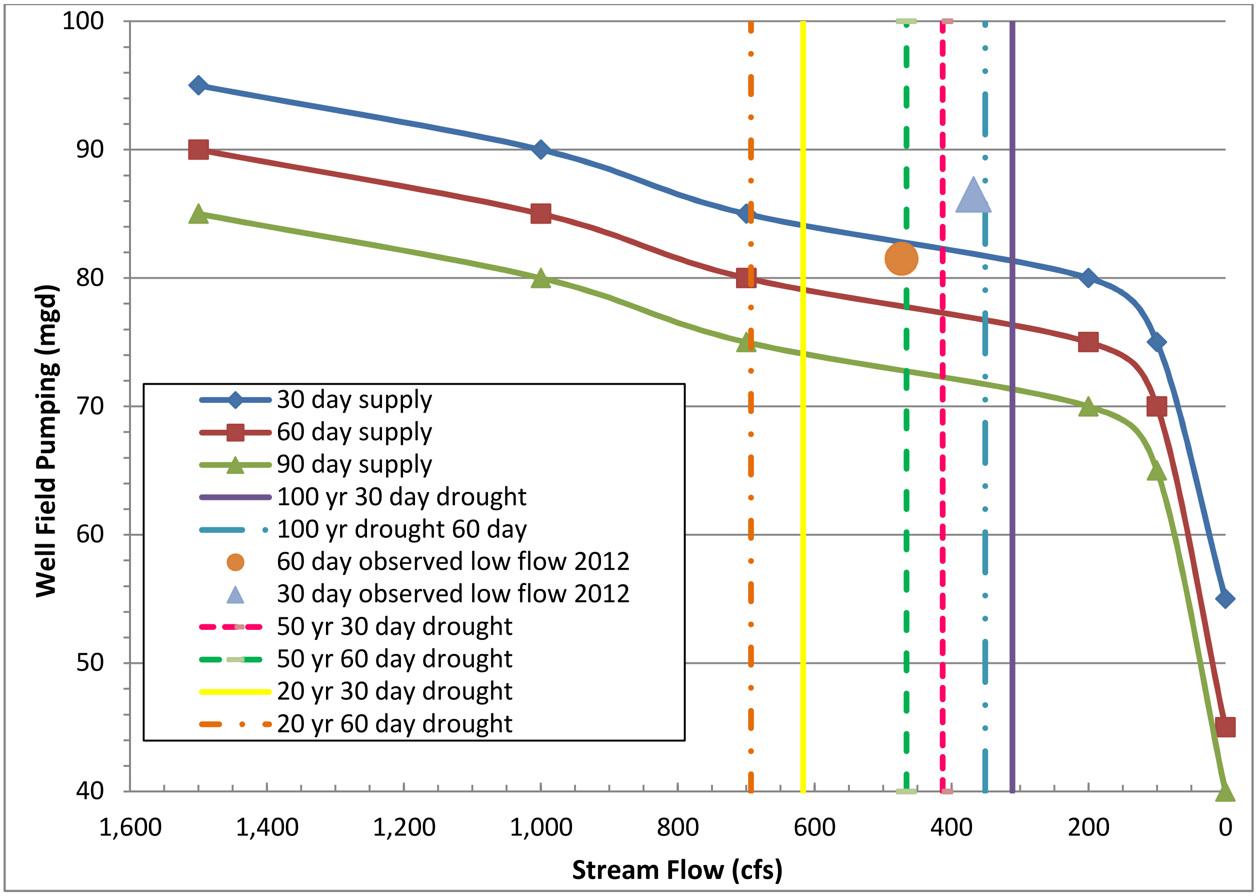


Figure 3-3 Model-Predicted Well Field Capacity with third HCW and Drought Reoccurrence Interval Window

As shown in Figure 3-3, the yield of the City's well field is variable and is a function of streamflow in the Platte River and the duration of high volume pumping. However, Figure 3-3 shows that for planning purposes, there is a small difference between what the well field can supply over a relatively large range of streamflow values and pumping durations.

3.6 Supply Deficit

Results of the updated groundwater model indicate that the 60- to 90-day production capacity of the City's well field ranges from 75 to 80 mgd when streamflow is less than 1,000 cfs and from 71 to 77 mgd during a streamflow event that correlates to a 100-year reoccurrence interval drought. This production capacity is then compared to the water demand forecasts that were developed in *Chapter 2 - Water Capacity Requirements* to determine the supply deficit. This evaluation is shown graphically in Figure 3-4.

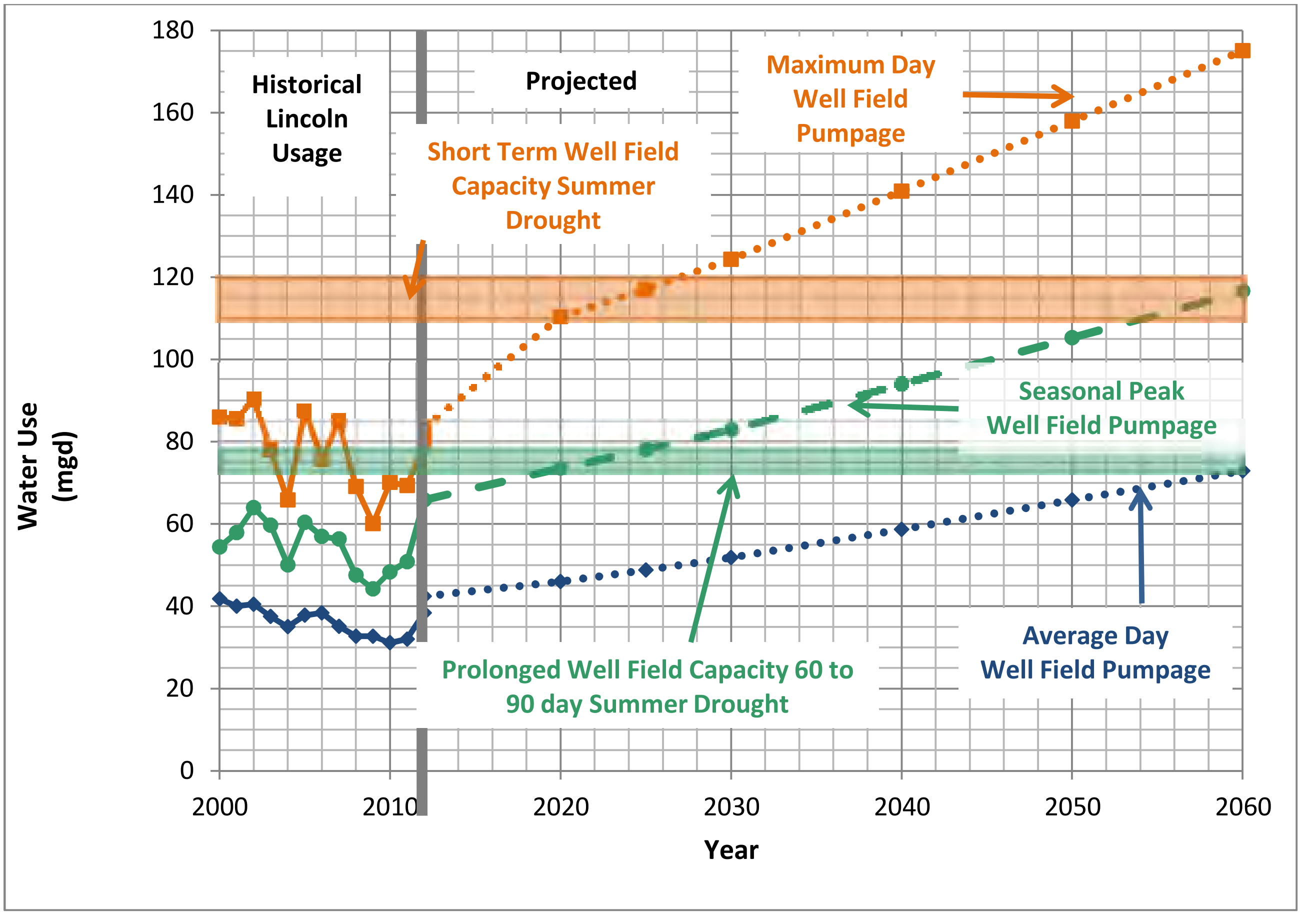


Figure 3-4 Raw Water Supply Deficits over Planning Horizon (with the Third HCW)

Through interpretation of this figure, the following conclusions can be developed:

- Once the third HCW is available, the well field would have sufficient instantaneous and short-term pumping capacity to meet the maximum day demand during extreme summer drought and low-flow conditions until approximately 2022.
- The projected seasonal peak demand exceeds the supply capacity during extreme summer drought and low-flow conditions in 2018 even with the third HCW available.
- The supply difference between the projected seasonal peak in 2060 demand and the water that can be produced by the well field during prolonged drought conditions is between 35 and 45 mgd.
- The supply difference between the projected maximum day demand in 2060 and the short-term water production rate that can be sustained by the well field is between 50 and 60 mgd.

4.0 Alternative Supply Analysis

This section evaluates raw water supply alternatives that would increase the raw water capacity of the City to meet both the short-term and long-term demands. An additional consideration of this analysis was the desire of the City to increase the reliability of the raw water supply by diversifying the raw water source. Based on the analysis presented in Section 3.0, the City's well field, in its current configuration, will not be able to meet projected demands through the planning horizon of 2060. The previous analysis identified potential short-term and long-term deficits in raw water supply, even when including the third HCW. Three planning horizons were identified for this evaluation, as defined below:

Short-term horizon (2014 to 2025)

During periods of prolonged low streamflow in the Platte River, the projected water demand could exceed the 60- to 90-day pumping capacity as early as 2018 depending on the magnitude and duration of a drought. In 2018, a supply deficit would be anticipated to occur only during extreme drought conditions that correlate to the 50- to 100-year reoccurrence interval event. By 2025, a supply deficit would be anticipated to occur during more frequent drought events, such as the 20-year reoccurrence interval event. There is also a projected supply deficit with the instantaneous and short-term pumping capacity of the well field, where it is projected that the well field may not be able to meet the maximum day demand as early as 2022 during times when the 1-day streamflow is less than the 50- to 100-year reoccurrence interval drought.

The short-term supply deficits projected in Figure 3-4 are relatively small. The addition of 20 mgd of instantaneous and short-term water supply and 10 mgd of supply that can be sustained for 60 to 90 days would meet the projected water supply needs through approximately 2030. Therefore, the objective of the short-term supply evaluation is to develop a recommendation to expand the existing source of supply, as described above.

Mid-term horizon (2026 to 2040)

The projected water demands in 2040 are a seasonal peak of 95 mgd and an instantaneous short-term pumping rate of 137 mgd. The supply difference between these projected values and the water that can be currently produced by the well field during prolonged drought conditions is approximately 15 to 25 mgd. The supply difference between the projected maximum day demand in 2040 and the instantaneous and short-term water production rate that can be supplied by the well field during periods when the streamflow is less than 1,000 cfs is also between 15 and 25 mgd.

Long-term horizon (2041 to 2060)

The projected water demands in 2060 are a seasonal peak of 115 mgd and an instantaneous short-term pumping rate of 170 mgd. The supply difference between the projected seasonal peak in 2060 and the water that can be currently produced by the well field during prolonged drought conditions is between 35 and 45 mgd. In this case, the prolonged drought equates to an event that has a reoccurrence interval of 100 years. The supply difference between the projected maximum day demand in 2060 and the instantaneous and short-term water production rate that can be supplied by the well field during periods when the streamflow is less than 1,000 cfs is between 50 and 60 mgd.

4.1 Short-term Supply Options (2014 to 2025)

The analysis presented in this section builds upon the previous alternative analysis completed by LWS and includes an analysis of alternatives that were not previously evaluated. The short-term supply deficits presented in Figure 3-4 are relatively small. Based on the analysis of seasonal well field capacity, it appears that the addition of 15 mgd of instantaneous and short-term water supply and 10 mgd of supply that can be sustained during a summer drought would meet the projected water demands through 2030. However, a potential supply deficit between projected demand and available supply could occur as early as 2020; therefore, the short-term supply option must be permitted, designed, and constructed, and must start operating prior to 2020. Consequently, only options that can be implemented near term were evaluated as methods to increase the raw water supply. The options evaluated are listed below:

1. Expansion of existing well field with completion of the fourth HCW
2. New well field in the High Plains Aquifer
3. Aquifer storage and recovery (ASR) as peak shaving
4. Metropolitan Utilities District (MUD) interconnection
5. Water reuse
6. Conservation (Given the uniqueness of this option, it is presented as a stand-alone evaluation in Section 6.0, Water Conservation.)

4.1.1 Expansion of Existing Well Field (Fourth HCW) Option

The projected water demands in 2060 are a maximum day of 175 mgd and a seasonal peak of 115 mgd. It is unclear if the development of this type of high-volume well field pumping can be sustained from the aquifer that underlies the permitted diversion area, as permitted by water rights (see Figure 3-1). Determination of the feasibility of this type of well field expansion can be made only through the use the well field groundwater flow model and would require model runs for each specific development scenario. Previous model runs that evaluated the expansion of the City's Well Field indicated that the total sustainable well field yield ranges from 92 mgd with streamflow at 1,000 cfs to 94 mgd with streamflow at 3,000 cfs (B. Kroeker of TZA Water Engineers, Inc., personal communication, October 11, 2013). For this modeling evaluation, the east bank of the well field consisted of five HCWs and four vertical wells.

Although it is unclear if expansion of the existing well field is a viable option to meet the projected 2060 demands, expansion of the existing well field is a viable method to meet the short-term supply deficits identified in Figure 3-4, which are relatively small. The deficit in raw water supply identified above could be addressed through the construction of a fourth HCW.

The updated well field capacity, including the fourth HCW is presented in Table 4-1. For this table, it was assumed that the fourth HCW would provide 15 mgd of instantaneous and short-term supply and 10 mgd of supply during a 60- to 90-day drought.

As shown in Figure 4-1, adding the fourth HCW to the well field supply ensures that both the maximum day and seasonal peak projected demands are met until approximately 2030.

Table 4-1 City's Well Field Seasonal Firm Capacity with the Fourth HCW

Season	Streamflow (cfs)	Water Level Conditions	Maximum Instantaneous Pumping (mgd)	Maximum Pumping for 2 Months (mgd)	Maximum Pumping for 3 Months (mgd)
Spring	3,000 or greater	High	145	120	115
Summer low flow	1,000 to 500	Low	135	90	85
Summer drought	< 500 (50 to 100 Year Reoccurrence Interval Event)	Low	125	87	81

Note:
The 100–year, 60-day average streamflow during drought at Ashland is 351 cfs. The 100–year, 90-day average streamflow during drought at Ashland is 465 cfs.

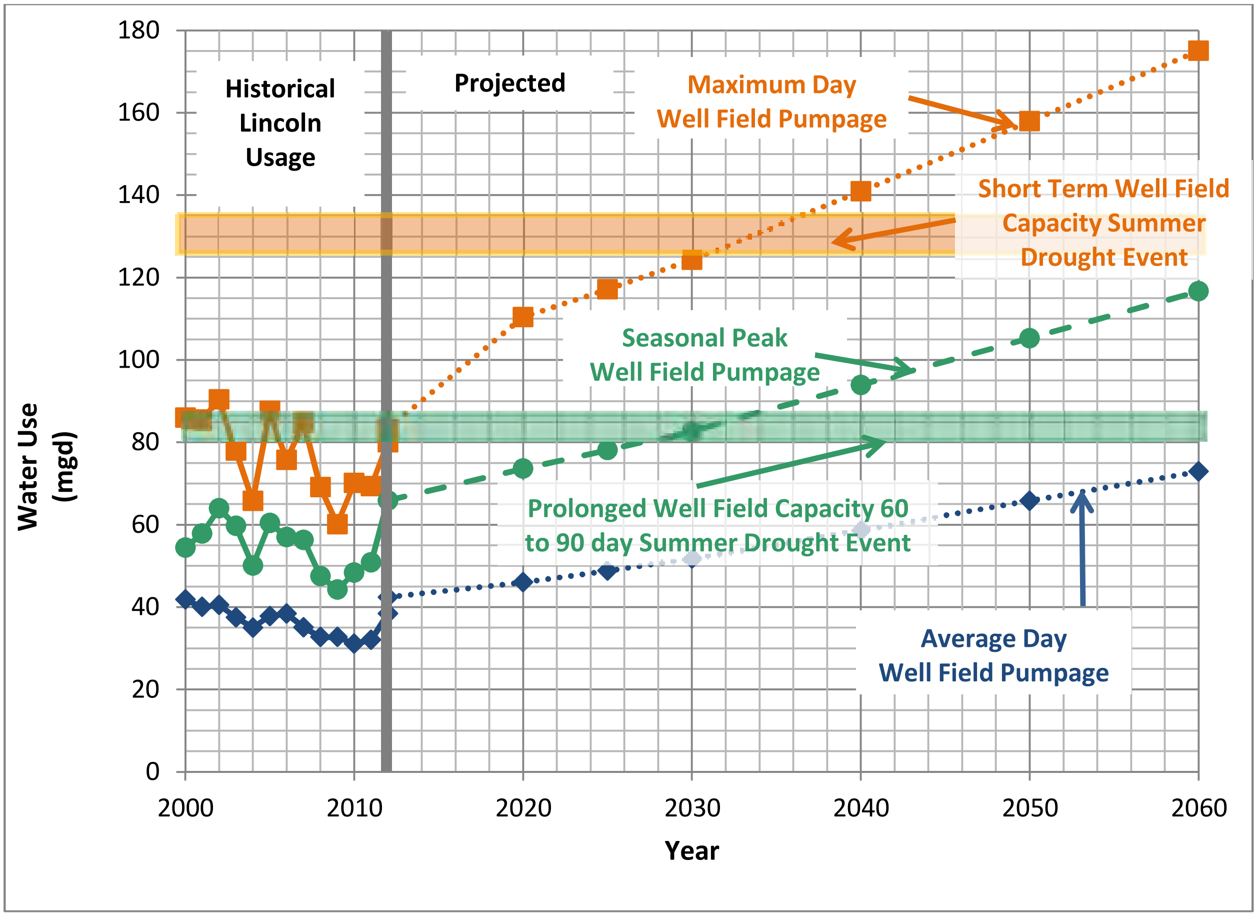


Figure 4-1 Raw Water Supply with the Fourth HCW and Revised Deficits over Planning Horizon

Costs to construct the fourth HCW are presented below in Table 4-2. The costs and quantities for all items other than the river crossing were obtained from bid tabs provided by LWS. The cost for the river crossing was obtained from information presented in the *Water Supply Upgrades Horizontal Collector Wells* (Black and Veatch 2013).

Table 4-2 Cost Estimate to Construct the Fourth HCW

Item	Quantity	Unit	Price	Total
Well house, pumps, transmission pipelines, and roadway grading	1	Lump Sum	$6,100,000	$6,100,000
River crossing	1	Lump Sum	$4,200,000	$4,200,000
			Total	$10,300,000

Note:
Costs presented are in 2013 dollars.

4.1.2 New Well Field in the High Plains Aquifer Option

Development of a new well field in the High Plains Aquifer system was evaluated as an alternative for providing a water supply that could address the short-term deficit identified. In Nebraska, the High Plains Aquifer consists of Pleistocene sand and gravel deposits that overlie the Ogallala Formation. Because the Ogallala Formation is not available within 50 miles of Lincoln, this evaluation focused on only the sand and gravel deposits that overlie bedrock.

Figure 4-2 presents a regional saturated thickness map of the sand and gravel deposits in the High Plains Aquifer west of Lincoln. This map was developed by HDR as part of a groundwater modeling study of the Blue River Basin that was developed for NDNR. As shown in Figure 4-2, a number of areas within the sand and gravel deposits of the Blue River Basin have a sufficient thickness of saturated sand and gravel to support high-capacity municipal wells. Wells could be developed in areas where the sand and gravel depositions of the High Plains Aquifer intersect the alluvium of the Big Blue River to induce recharge from the river. Alternatively, wells could be developed away from the Big Blue River in areas where recharge from the surface water body is limited.

The closest High Plains Aquifer deposits are located in the Blue River Basin. For this option, the feasibility of developing a groundwater supply in the High Plains Aquifer west of York, Nebraska, was evaluated.

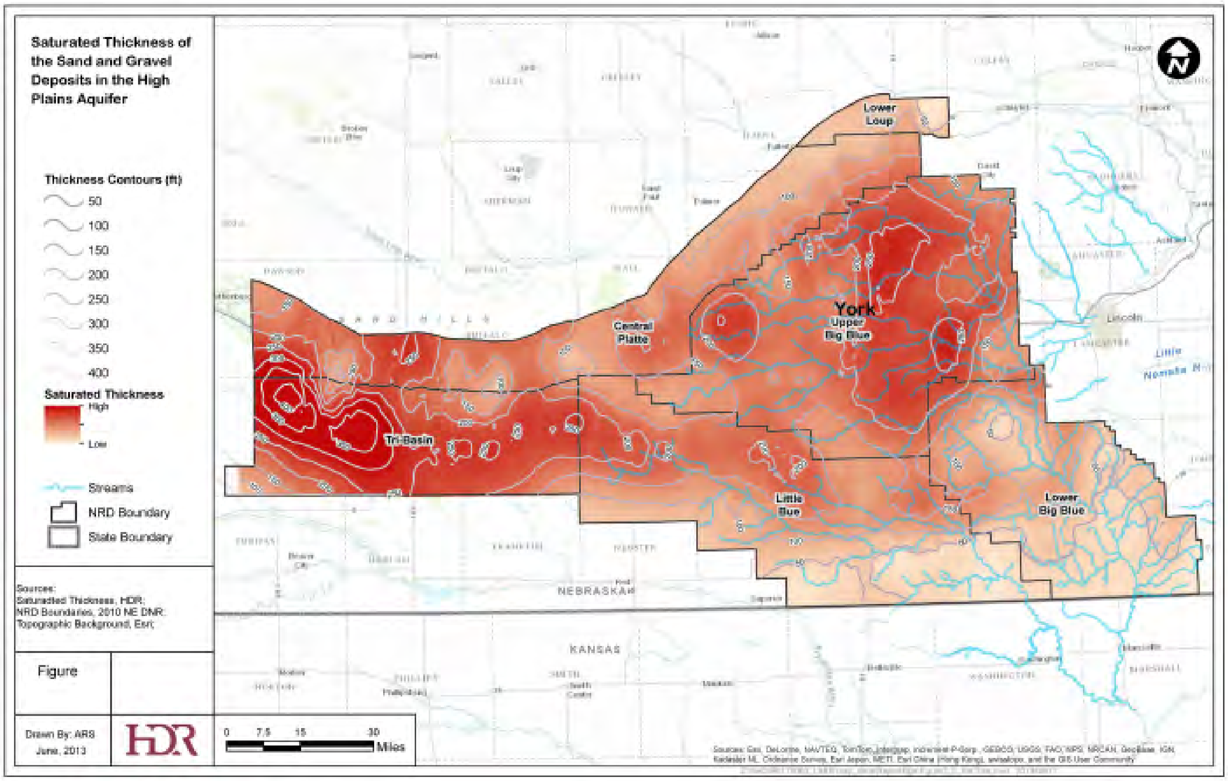


Figure 4-2 Saturated Thickness of High Plains Aquifer West of Lincoln

Figures 4-3a and 4-3b illustrate the geology that is typical of the Blue River Basin. The sand and gravel deposits are variable, and the thickness of these deposits is controlled by the bedrock elevation, which is variable. Section E-E’ in the figures illustrates the geology that would most likely be encountered if a well field were developed in the High Plains Aquifer west of York.

As shown in Figure 4-2, the saturated thickness of the sand and gravel deposits of the High Plains Aquifer ranges from 150 to 200 feet west of York. Well yields of 500 to 1,000 gpm are common for irrigation wells. Figures 4-3a and 4-3b illustrate the variability of the aquifer and the water table (unconfined) conditions that exist in these deposits.

8 Hydrogeology and Subsurface Nitrate in the Upper Big Blue Natural Resources District, Central Nebraska, July 1995 through September 1997

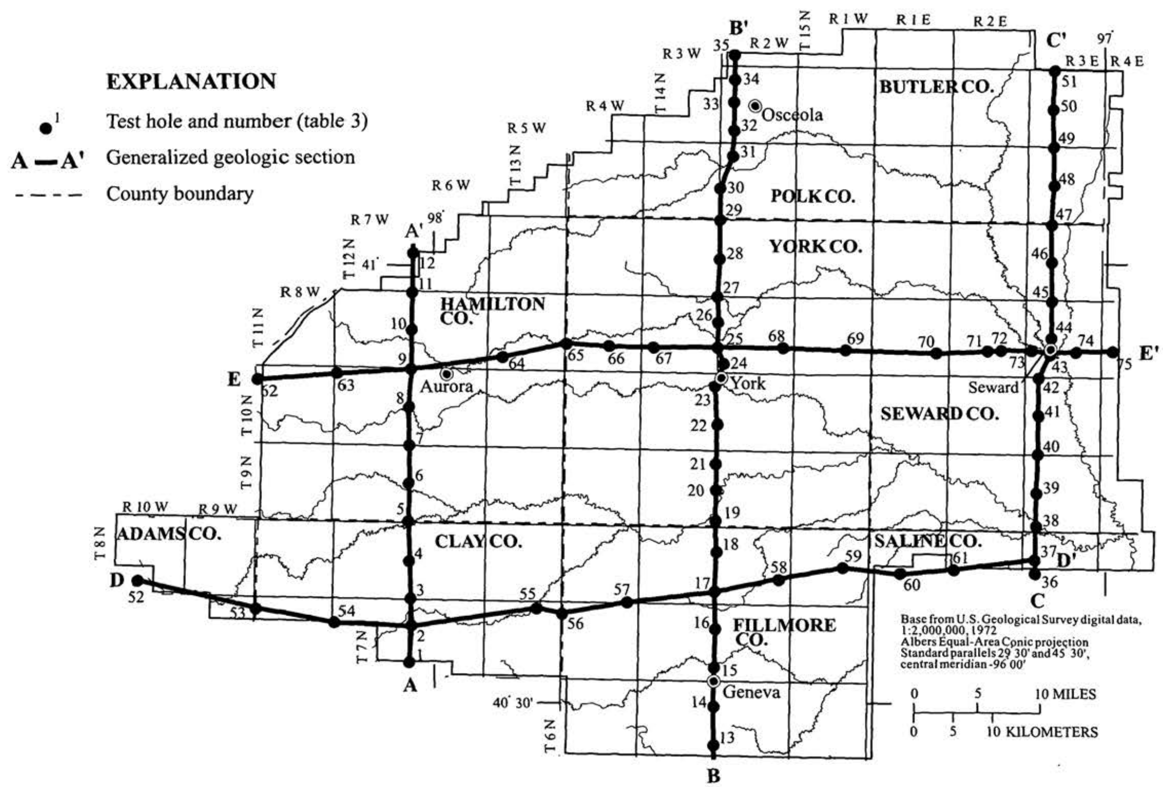


Figure 3. Locations of test holes and generalized geologic sections (figure 4).

Source: Hydrogeology and subsurface nitrate in the Upper Big Blue Natural Resources District, central Nebraska, July 1995 through September 1997 (USGS 1998).

Figure 4-3a Geologic Cross Section Location Map

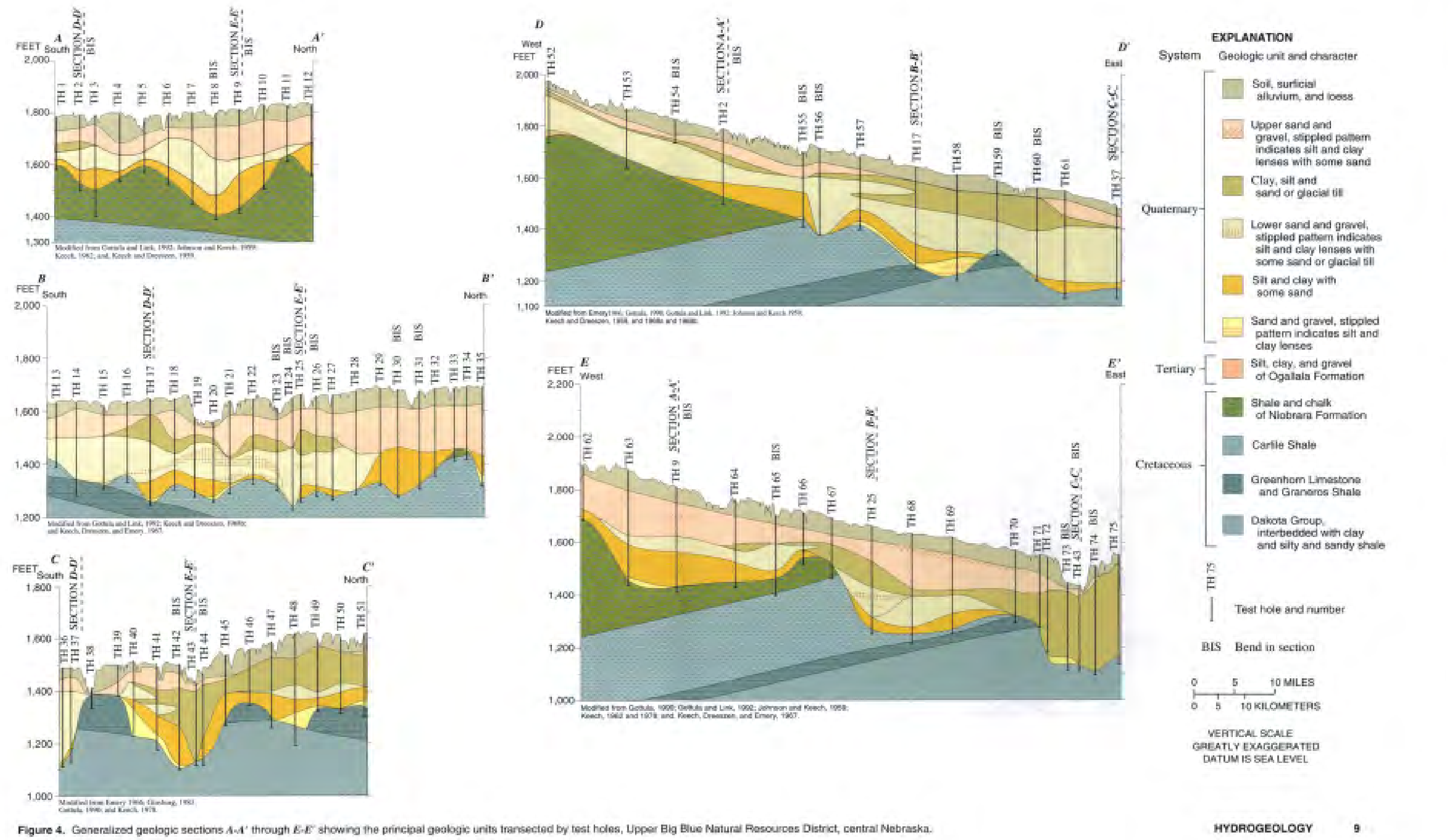


Figure 4. Generalized geologic sections A-A' through E-E' showing the principal geologic units transected by test holes, Upper Big Blue Natural Resources District, central Nebraska.

Source: Hydrogeology and subsurface nitrate in the Upper Big Blue Natural Resources District, central Nebraska, July 1995 through September 1997 (USGS 1998).

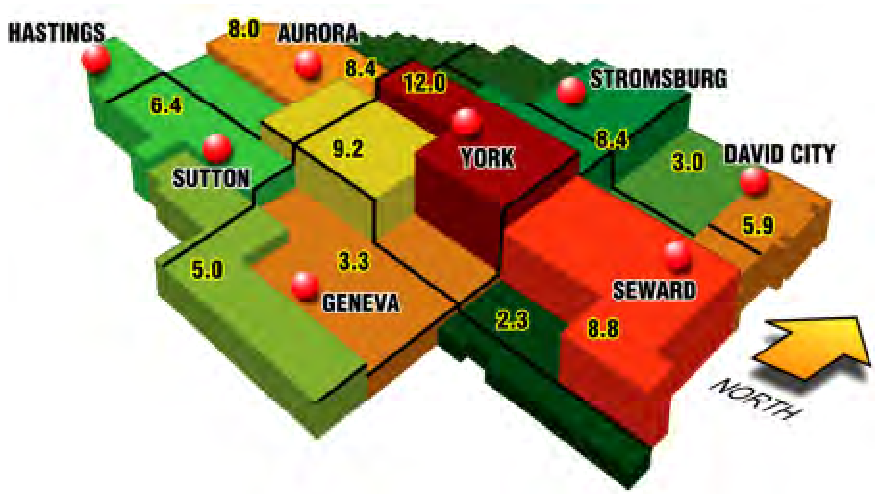
Figure 4-3b Geologic Cross Sections

Permitting

A new well field constructed in the sand and gravel deposits of the High Plains Aquifer would require permitting through the local Natural Resources District (NRD). For a well field west of York, the local NRD is the Upper Big Blue NRD (UBBNRD). UBBNRD has required that all new water wells be permitted. If the total withdrawal for that well is more than 500 acre-feet (0.4 mgd), UBBNRD requires a Large Water User Study, which is designed to evaluate the sustainability of the proposed well development. These Large Water User Studies typically require the development of a groundwater flow model. Given the potentially large volume associated with a new well field, it is anticipated that gaining approval for a relatively large municipal well field in the Big Blue River Basin would be challenging.

Water Quality

UBBNRD regularly monitors groundwater quality within the area being considered as a potential water supply alternative for the City. The primary contaminant that is monitored is nitrate. A total of 12 nitrate management zones have been developed as part of this effort. Results from this monitoring effort indicate that nitrate contamination within the sand and gravel aquifer is a significant issue. Specifically, the management zones that include the cities of Seward, Nebraska, and York had median nitrate concentrations of 8.1 parts per million (ppm) and 12.0 ppm, respectively. The maximum contaminant level for nitrate in drinking water is 10 milligrams per liter (mg/L). As a result, some level of treatment is likely needed for nitrate reduction, which typically requires advanced water treatment processes such as reversed osmosis. Results for all management zones are presented below in Figure 4-4.



Note: Concentrations in ppm
Source: http://www.upperbigblue.org/nrdwebsite/blueprints/blueprintjuly11.pdf

Figure 4-4 2010 Median Nitrate Concentrations in Groundwater for UBBNRD Management Zones 1 through 12

USGS evaluated groundwater contaminants in the sand and gravel deposition of the High Plains Aquifer in the *Simulations of Ground-Water Flow, Transport, Age, and Particle Tracking near York, Nebraska, for a Study of Transport of Anthropogenic and Natural Contaminants (TANC) to Public-Supply Wells* (USGS 2007). This report identified arsenic and uranium as naturally occurring groundwater contaminants that are found in this aquifer on a regional scale.

Summary – New Well Field in the High Plains Aquifer

Due to the numerous issues regarding groundwater quality in the sand and gravel deposits of the High Plains Aquifer, this option was not considered as a viable alternative for the City. A cost estimate was not developed for this option.

4.1.3 Aquifer Storage and Recovery Peak Shaving Wells Option

Aquifer storage and recovery (ASR) is the enhancement of natural groundwater supplies using human-made conveyances, such as infiltration basins or injection wells, with the purpose of both augmenting groundwater resources and recovering the water in the future for various uses. ASR wells are used to achieve two objectives: 1) storing water in the ground; and 2) recovering the stored water either using the same well or by pairing injection wells with recovery wells located in the same well field. This section describes the development of ASR wells in the Cretaceous Dakota Aquifer (Dakota Formation). The ASR concept is evaluated as an alternative water supply during periods of peak demand.

The Dakota Formation consists of fine- to medium-textured sandstone interbedded with shale and approaches 140 feet in thickness in some areas near the City. Near Lincoln, the Dakota Formation is overlain by Quaternary deposits of loess, till, or sand and gravel, and is generally an unconfined to semi-confined aquifer that can be hydraulically connected to the overlying shallow sands and units. The groundwater in the Dakota Formation near the City is of poorer quality than water from overlying sand and gravel aquifers (USGS 1994). The water quality from the Dakota Formation is highly variable, with reported dissolved-solids concentrations ranging from 23,700 to 43,800 mg/L (USGS 1994). The yield from wells constructed in the Dakota Formation is also highly variable and ranges from 50 to 750 gpm (USGS 1994).

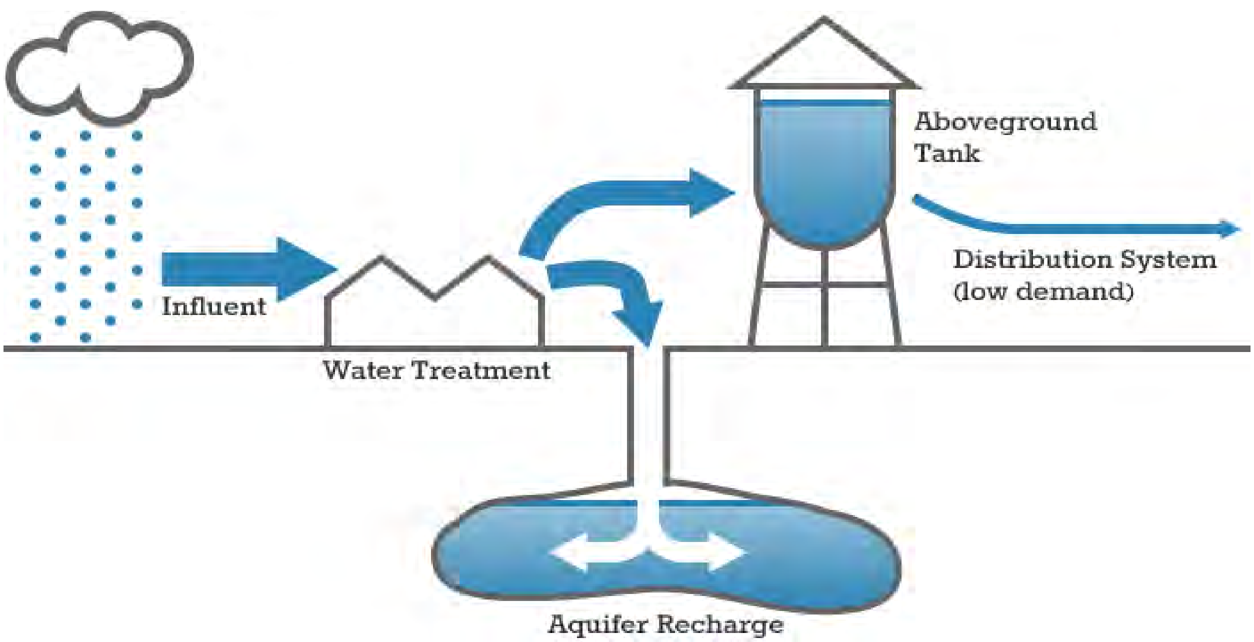
The concept evaluated in this Master Plan is to develop a number of ASR wells near existing City infrastructure, such as large transmission mains and electrical power, and to use the ASR wells as a peak shaving supply during periods of extended drought. MUD uses wells constructed in the Dakota Formation as a peak shaving supply to supplement water from other sources. The only treatment for the peak shaving wells is disinfection at the well head. A similar concept is evaluated for the City; however, the water quality of the Dakota Formation in the City is poor compared to the water quality of the Dakota Formation near Omaha, Nebraska.

The poor water quality of the Dakota Formation near the City requires the use of ASR if this aquifer is to be used. A description of the ASR concept is provided below.

4.1.3.1 ASR as a Peak Shaving Tool

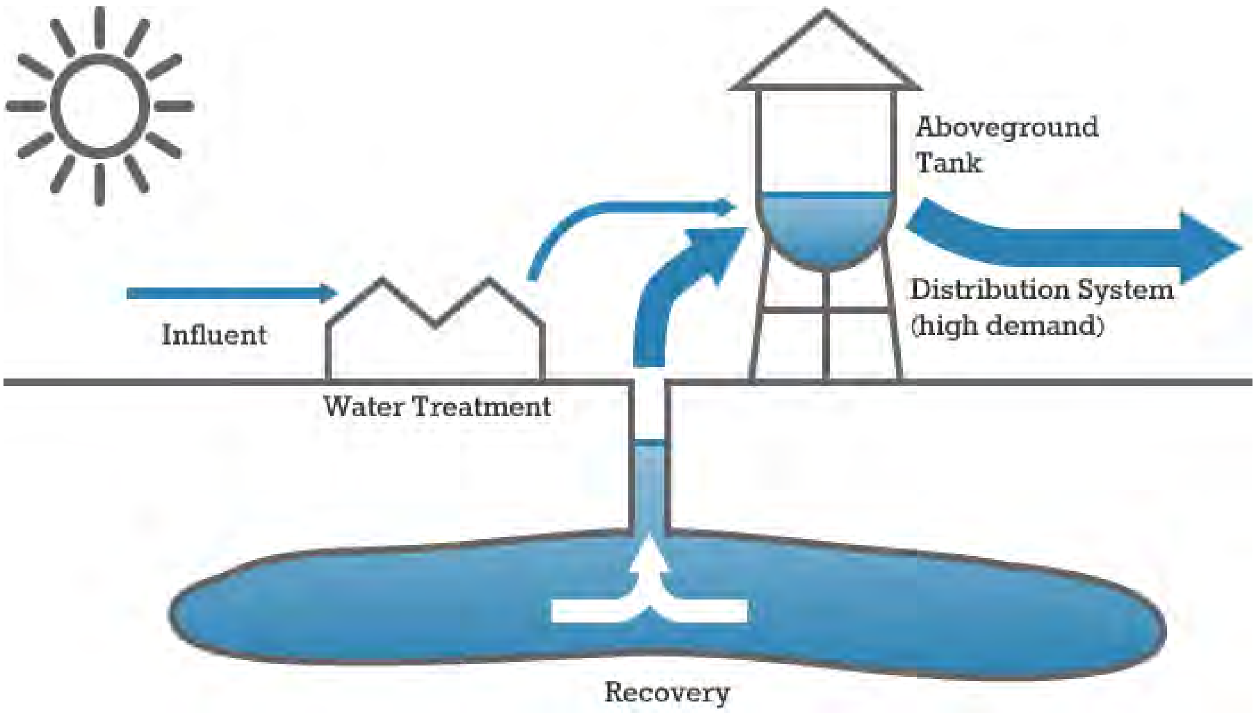
Dual purpose (that is, injection and extraction) ASR wells could be located strategically around the City near existing water transmission infrastructure. These wells would be located within 500 feet to 1,000 feet from a large transmission main and available electrical service, to limit the capital cost. Treated water would be injected into the Dakota Formation through the dual purpose wells during periods of low water demand and excess water treatment capacity. Over time, a bubble of treated water will develop near the ASR well. This treated water can be pumped out of the aquifer during periods of high water demand. The ASR wells could be developed into what would effectively become a second water supply source that could be used during a period of drought.

Figures 4-5 and 4-6 illustrate the concepts of ASR injection and recovery cycles, respectively. The recovery efficiency of an ASR system is defined as the percentage of the water stored that is subsequently recovered. In the Dakota Formation, monitoring of the total dissolved solids (TDS) in the recovered water during a recovery cycle will provide a clear indication of when the well is pumping treated water, mixed water, or formation water. As described above, the native water in the Dakota Formation near the City has very high TDS. Without detailed pilot testing, it is not possible to estimate the recovery efficiency of an ASR well; however, a typical range is 80 to 100 percent (Pyne 1995).



Source: http://indewater.com/aboutasr/

Figure 4-5 ASR Injection Cycle



Source: http://indewater.com/aboutasr/

Figure 4-6 ASR Recovery Cycle

4.1.3.2 Capacity

Well yield in the Dakota Formation is highly variable. For the purpose of this study, the pumping (during recovery phase) yield for a Dakota Formation well was assumed to be 500 gpm (0.72 mgd). To achieve the 20 mgd short-term supply capacity goal, approximately 30 dual purpose ASR wells are required.

4.1.3.3 Permitting

ASR wells are regulated as Class V injection wells by the Nebraska Department of Environmental Quality (NDEQ). As such, ASR well owners and operators are required to submit an Underground Injection Control (UIC) permit to NDEQ. The injection water must meet drinking water standards and must not degrade the water quality of the aquifer. A UIC permit can typically be obtained through NDEQ provided the operator can demonstrate the above. Other permits required would be similar to permitting a water supply well in the Lower Platte South NRD (LPSNRD).

4.1.3.4 Cost Estimating

A cost estimate was developed to implement the ASR concept. The cost estimate was developed assuming that a total of 30 ASR wells would be constructed, each with a 500 gpm pumping capacity. It was assumed that the wells would each be 150 feet deep and would be constructed using 18-inch-diameter screen and casing. Additionally, it was assumed that a well house equipped with an at-the-well chlorination system, consisting of two, 100-pound chlorine gas cylinders located in the well house, would be constructed for each well. Given the variability of the Dakota Formation, an ASR pilot testing program was also included. An estimate of capital costs is presented below.

Table 4-3 Cost Estimate for ASR Peak Shaving Wells

Item	Quantity	Unit	Price	Total	Assumptions
ASR wells and pumps	30	Each	$200,000	$6,000,000	18-inch well and line shaft turbine pump
Well house and disinfection system	30	Each	$150,000	$4,500,000	Disinfection using gas cylinders located inside the well house
12-inch well piping	15,000	Linear Foot	$120	$1,800,000	500 feet of piping at each site
Pilot testing	1	Lump Sum	$1,000,000	$1,000,000	Assumes pilot testing at 2 well locations
Contingency	25%			$3,250,000	
Engineering	15%			$2,000,000	
			Total	$18,250,000	

Note:
Based on 2013 dollars

4.1.4 Interconnection with Metropolitan Utilities District Option

Two interconnections with MUD's system were considered as options for additional supply. These included:

1. A raw water interconnection between the MUD Platte West well field and the City's well field
2. A finished water interconnection between the MUD distribution system and the LWS finished water piping at the Platte River Water Treatment Facility

4.1.4.2 Raw Water Interconnection

The raw water interconnection analysis considered a new pipeline to connect the MUD well field to the LWS well field. The MUD Platte West well field is located on either side of the Platte River in Douglas and Saunders counties upstream of the City's well field. The southern end of the MUD well field is about 7 miles north of the northern end of the City's well field. The existing MUD well pumps are able to provide sufficient head to convey about 12 mgd raw water through a 24-inch-diameter interconnecting transmission main. This interconnection would continue the

City's reliance on the Platte River as a source of supply but would potentially provide a larger pool from which to withdraw water.

The estimated cost for the interconnecting raw water pipe is presented in Table 4-4.

Table 4-4 Cost Estimate for Raw Water Interconnection with MUD

Item	Quantity	Unit	Price	Total
24-inch transmission main	36,960	Linear Foot	$240	$8,870,400
Contingency	25%			$2,217,600
Engineering	15%			$1,331,000
			Total	$12,419,000

The water quality from this raw water interconnection would be anticipated to be approximately the same as the water quality from the City's well field. As a result, this water could be pumped to the Platte River Water Treatment Facility for treatment.

An Environmental Impact Statement (EIS) was prepared as a part of the planning and design of the MUD Platte West well field. The use of the raw water as a source of supply for the City could potentially impact this EIS and would have to be further evaluated prior to developing this supply option.

4.1.4.3 Finished Water Interconnection

The option considering a finished water interconnection included a new pipeline connection to the MUD distribution system. Due to the 16.5-mile length of the route, a 30-inch-diameter transmission main is required to minimize the cumulative friction losses in the pipe. The elevation of the MUD distribution system at this location is about 110 feet higher than the Platte River Water Treatment Facility. The available static head and the head provided by the MUD Platte West water treatment plant Zone 2 high service pumps are sufficient to convey about 15 mgd of finished water to the Platte River Water Treatment Facility. The interconnecting finished water main route crosses the Platte River, and it is assumed that the river crossing would be installed by horizontal directional drilling.

The finished water quality parameters of the LWS finished water and the MUD Platte West finished water were compared to evaluate whether the water qualities were compatible. The pH and Langelier Index (LI) of both waters and the blended water are summarized in Table 4-5.

Table 4-5 pH and Langelier Index (LI) of LWS, MUD, and Blended Water

Water	pH	LI
LWS	7.6	0.15
MUD	8.9	1.21
Blended[1]	7.7	0.20

Note:

1. Blended water calculations are based on a flow of 60 mgd of LWS water and 15 mgd of MUD water.

The LWS finished water is slightly depositing, and the MUD finished water is more depositing. Based on the blended water pH of 7.7 and the blended water LI value of 0.20, it is expected that blending MUD water to the LWS finished water supply would not change the LWS water quality significantly and would not cause the water to become corrosive or appreciably more depositing.

At the termination of the finished water transmission main at the Platte River Water Treatment Facility, it is recommended that the transmission main feed into a new finished water storage tank. From the new storage tank, the water can be gravity fed into the suction side of the LWS high service pumps. In addition to providing additional storage capacity during periods of high demand, the tank would allow the MUD water to be metered into the LWS finished water line at a controlled rate and provide a blending for consistent water quality. Table 4-6 presents the estimated costs for the finished water interconnection.

Table 4-6 Cost Estimate for Finished Water Interconnection with MUD

Item	Quantity	Unit	Price	Total
30-inch transmission main	87, 120	Linear Foot	$300	$26,136,000
River crossing[1]	1	Lump Sum	$1,900,000	$1,900,000
New 3.0 MG Storage Reservoir at Platte River Water Treatment Facility	1	Lump Sum	$1,800,000	$1,800,000
Contingency	25%			$7,500,000
Engineering	15%			$4,500,000
			Total	$41,836,000

Note:

1. 1,400 LF 30-inch directional drill installed finished water transmission main

2. Based on 2013 dollars

The raw water interconnection option is estimated to be about one third of the cost of the finished water interconnection option. However, the reliability of a raw water connection to a well field that takes water from the same aquifer as the LWS well field is lower than a finished water connection, which is supplied from additional water sources. Additional discussions between LWS and MUD are required to further evaluate the feasibility of raw water and finished water interconnections.

4.1.5 Water Reuse Option

The City is already capitalizing on reuse as a water supply source. A system is currently under design to use effluent from the City's Theresa Street Wastewater Treatment Plant (WWTP) for heating and cooling at the new Nebraska Innovation Campus. The City believes that within approximately 20 years, the Nebraska Innovation Campus could potentially reuse all of the effluent flow from the 27-mgd Theresa Street WWTP. The City's Northeast WWTP sends about 40 million gallons of effluent per year to the Lincoln Electric System Terry Bundy Generating Station. The effluent flow to the generating station is seasonal when Lincoln Electric System uses the station to meet peak loads in the summer. The Northeast WWTP is a 9.9 mgd plant, so additional effluent flow from this facility is available to be reused by other industries in the area or possibly for irrigation purposes but would not be of sufficient capacity to meet the short-term supply option needs. The City's WWTPs disinfect from May through September, so potential irrigation usage would need to be coordinated with the periods of time the Northeast WWTP effluent is disinfected.

4.2 Mid-Term Supply Options (2026 to 2040)

A deficit between the projected seasonal peak water demand and the raw water that can be produced by the existing wells (including the third and fourth HCW) could occur as early as 2025. By 2040, this deficit in seasonal peak water supply production capacity will range from 5 to 15 mgd, depending on streamflow conditions. A deficit in the instantaneous pumping capacity of the well field compared to the maximum day demand is projected to occur in 2032. By 2040, this deficit in maximum day production capacity is projected to be 5 to 15 mgd, also depending on streamflow conditions. Figure 4-1 highlights the projected water supply demand against the well field production capacity (including the third and fourth HCW).

Two mid-term water supply options were evaluated to meet the projected supply deficit from 2026 to 2040. These options include the expansion of the existing well field and the development of a surface water reservoir.

4.2.1 Well Field Expansion Option

A plan to expand the existing well field was presented in the *Lincoln Water Systems Facilities Master Plan* (Black and Veatch 2003). The 2002 Master Plan consisted of developing two additional HCWs and a total of 13 additional vertical wells. The 2002 Master Plan stated that this expansion would increase the well field seasonal yield to 125 mgd and the firm capacity of the well field to 220 mgd.

Groundwater model runs performed following the 2002 Master Plan evaluated the expansion of the City's Well Field. In this evaluation, model simulations were performed with a well field consisting of 40 vertical wells and 6 HCWs. Under this well field configuration, the model estimated that the well field could produce 111 mgd for 60 days and 107 mgd for 90 days with a streamflow of 200 cfs. These modeled values compare favorably to the values estimated using conservative summer HCW pumping rates of 10 mgd for the new HCWs.

Assuming each new HCW will increase the summer seasonal well field yield by 10 mgd and the maximum instantaneous pumping rate by 15 mgd, a fifth HCW would increase the summer seasonal pumping capacity of the well field to between 91 and 97 mgd during drought conditions. This pumping capacity would meet projected seasonal demands to 2035. The addition of a sixth HCW would increase the summer seasonal pumping capacity of the well field to between 101 and 107 mgd during drought conditions, a number that compares favorably to the values estimated using the model. This pumping capacity would meet projected seasonal demands to approximately 2045.

The addition of these two new HCWs would increase the maximum instantaneous pumping capacity of the well field to 175 mgd (155 mgd during drought), as shown in Table 4-7. These additional HCWs would address peak day demand conditions during a drought through approximately 2050. The fifth HCW should be operational by 2025 and would address projected increases in water demand to 2035. The sixth HCW should be operational before 2035 and would address projected seasonal demands to approximately 2045 to 2050. As discussed above in the raw water transmission section, the addition of the sixth HCW will require the construction of a new raw water transmission main to connect the well to the Platte River Water Treatment Facility.

The estimated cost of the fifth HCW is $12.6 million in 2013 dollars, and the estimated cost of the sixth HCW and transmission main is $24.3 million in 2013 dollars. The HCW costs are based on the costs of a HCW presented in Table 4-2 with an additional $2 million for the caisson installation. The 48-inch raw water transmission main is estimated to cost $11.7 million in 2013 dollars.

Table 4-7 City's Well Field Seasonal Firm Capacity with Six HCWs

Season	Streamflow (cfs)	Water Level Conditions	Maximum Instantaneous Pumping (mgd)	Maximum Pumping for 2 Months (mgd)	Maximum Pumping for 3 Months (mgd)
Spring	3,000 or greater	High	175	140	125
Summer low flow	1,000 to 500	Low	165	110	105
Summer drought	< 500 (50 to 100 Year Reoccurrence Interval Event)	Low	155	107	101

Note:
The 100–year, 60-day average streamflow during drought at Ashland is 351 cfs. The 100–year, 90-day average streamflow during drought at Ashland is 465 cfs.

4.2.2 Reservoir Option

A second mid-term supply option identified was storage of raw water in a new or existing reservoir. For this option, excess water produced during periods of low demand would be stored in a surface reservoir until periods of high demand. For this evaluation, the expansion of an existing reservoir was used as an example to develop an understanding of the costs associated with developing surface water storage near the well field. For a reservoir to serve as an alternate source of supply, it would be required to provide water for a period of 30 to 90 days. In order for a reservoir to replace the construction of two HCWs as the mid-term supply option, it would have to provide up to 20 mgd of raw water during that period. It is estimated that a larger reservoir capable of storing approximately 1 billion gallons of water would cost in excess of $100 million.

4.3 Long-term Supply Options (2041 to 2060)

Once the third and fourth HCWs are constructed, the projected supply deficit in 2060 will be 50 mgd of instantaneous and short-term pumping capacity to meet the maximum day demand, and 35 mgd of summer yield capacity to meet the seasonal demand. However, if the mid-term supply option is constructed, the expanded City's Well Field could meet peak day demand conditions during a drought through approximately 2050 and projected seasonal demands to approximately 2045 to 2050.

In 2005, the City evaluated a number of options to address projected long-term water demands. Only two supply sources, the Missouri River and Platte River alluvial aquifers, were identified as viable options for the development of a large-scale, reliable water supply. An alternatives analysis presented in that study identified the Missouri River as the preferred water supply source for the additional supply for the following reasons:

- A supply located in the Missouri River basin would diversify the City's water supply sources, thereby increasing reliability.
- A new well field in the Platte River alluvial aquifer would need to be located sufficiently far enough away from the City's existing well field as to not impact or impair streamflow or groundwater elevations near the City's well field. To accomplish this, the new well field would have to be located at a significant distance from the existing well field, which increases the capital cost of this alternative.
- The Missouri River is operated as a navigable channel, and the streamflow is regulated from upstream reservoirs. A well field constructed in the Missouri River alluvium would be less susceptible to low streamflow during the summer months, when demands for water are highest.
- Permitting for the development of a new Platte River groundwater supply would require a much larger effort than development of a similar supply on the Missouri River.

The conclusions of the 2005 evaluation indicated that the Missouri River well field option would be able to support a 75-mgd demand even during significant drought conditions. Additionally, having separate sources of supply from the different rivers would result in source diversification of the raw water supply, which would provide operational flexibility to the City. Finally, although Platte River flows are their lowest in the summer months, the Missouri River is operated as a navigable channel and streamflow is therefore maintained at an artificially high level during periods of summer drought, resulting in significant recharge to a well field constructed in the alluvium of the Missouri River.

For the purposes of this Master Plan, it was assumed that all long-range options would supply a maximum of 60 mgd, which is sufficient to close the supply deficit identified for 2060 if one of the mid-term supply options is not developed. Should a mid-term option be constructed, development of a 60-mgd supply along the Missouri River would provide the City with a diversified source of supply that is resistant to drought and would provide the opportunity to develop a regional water supply.

5.0 Planning Level Permitting and Costs for Long-term Option

This section presents the permitting considerations and estimated capital costs for the long-term water supply option, Missouri River Project.

5.1 Permitting for Long-term Supply Option

Regulatory permits for the long-term option, Missouri River Project, were evaluated. A number of permits would be required to construct either option. A summary of the federal, state, and local permits is provided below.

Federal

- Section 404 of the Clean Water Act – The project may require coordination with USACE for authorization under Section 404 of the Clean Water Act. If the project does not impact (that is, dredge or fill) a water(s) of the U.S., no authorization is necessary. A wetland delineation is recommended to determine the presence of wetlands within the project area. If a Section 404 authorization is needed, the following additional requirements would need to be met:
    - Compliance with Section 7 of the Endangered Species Act (ESA) – USACE is responsible for compliance with the ESA. Depending on the potential for presence of federally listed threatened and endangered species, USACE may require habitat and/or species surveys. Given the project location, coordination may be needed for western prairie fringed orchid (*Platanthera praeclara*) and pallid sturgeon (*Scaphirhynchus albus*).
    - Compliance with Section 106 of the National Historic Preservation Act – USACE is responsible for compliance with Section 106 of the National Historic Preservation Act. This may require coordination with the Nebraska State Historic Preservation Office to obtain its opinion on the necessity for an on-site survey of the project area to document the potential for effects on historic properties.
- Section 408 authorization – Section 408 authorization would be required for construction in the critical area for a levee, which is generally considered to extend from 300 feet riverward to 500 feet landward of the levee centerline.
- No-Rise Certificate – Any construction in a floodway regulated by the Federal Emergency Management Agency (FEMA) will require a No-Rise Certificate showing that the improvements will not raise the 100-year flood frequency elevation of the river.

State

- NPDES General Stormwater Construction Permit – Administered by NDEQ, this permit is required for construction activities disturbing more than 1 acre of land. This permit requires coordination under the Nongame and Endangered Species Conservation Act to ensure that actions authorized, funded, or carried out by a state agency do not adversely impact state-listed species (Nebraska Revised Statute § 37 807(3)). The following species may be present in the project area that may need to be addressed with NDEQ and/or the Nebraska Game and Parks Commission for compliance:

  - American ginseng (*Panax quinquefolium*)
  - Lake sturgeon (*Acipenser fulvescens*)
  - Pallid sturgeon (*Scaphirhynchus albus*)
  - River otter (*Lutra canadensis*)
  - Southern flying squirrel (*Glaucomys volans*)
  - Sturgeon chub (*Macrhybopsis gelida*)
  - Western prairie fringed orchid (*Platanthera praeclara*)

- Well permits – Well permits would be coordinated through the local NRD.

Local

- Floodplain development permit – Coordination with local county authorities may be required for compliance with local floodplain regulations.

5.2 Cost Estimates for Long-term Supply Option

Planning-level capital cost estimates were developed for the Missouri River Project. The cost estimates include the well field infrastructure and associated transmission and treatment facilities. The capital costs are based on 2013 dollars. It was assumed that the long-range option would supply a maximum of 60 mgd, which is sufficient to close the supply deficit identified for 2060 if one of the mid-term options is not developed. It was assumed that a new 60 mgd treatment plant would be constructed on available land near the well field, and that the treatment plant would be constructed on the river bluff and not in the floodplain. Additionally, it was assumed that well pumps would be used to convey the water to the treatment plant. It was also assumed that the costs for upgrading the system within the proximity of the City would be approximately the same for all options, so the improvements within or near the City limits are not included in the capital costs.

Capital costs for treatment facilities and wells were developed using cost data from the existing LWS facilities as well as cost curves that were developed from a large number of local HDR projects that include new water treatment plants and well fields in the Missouri River alluvium. . Other cost assumptions are summarized below.

Cost Assumptions

- HCW investigation costs – based on 2012 HCW investigations performed by HDR.
- HCW costs – budgetary costs provided by Ranney Collector Wells (Layne 2013).
- Land costs – price based on Nebraska Farmer Real Estate Market Developments 2012–2013, Table 4: Average Reported Value of Nebraska Farmland, average price per acre of gravity irrigated cropland and center pivot irrigated cropland.
- Access roads – assumed 0.3 mile (1,584 linear feet) of road per well.
- Water treatment plant costs – estimated new treatment plant cost ranges from $1.71 per gallon per day for non-softening to $2.93 per gallon per day for softening, based on four new plants constructed within the past 5 years.
- Transmission main – 54-inch-diameter main price at a cost of $2.8 million per mile.
- Pump stations –Assumes $146,000 per mgd per pump station and reservoir; assumes pump station and reservoir every 10 miles.

Costs for the Missouri River Project ranges from approximately $500 million to $650 million depending on the length of transmission mains required to convey the water to the LWS distribution system.

6.0 Water Conservation

It is the policy of the City to promote water conservation. Water conservation encourages responsible use and preservation of the City's water supply. It also delays the need for expanding the City's existing water supply.

6.1 Existing Water Conservation Practices

Water Management Plan

The City has recently updated its Water Management Plan. The plan manages water use to maintain consumption within the system's production, pumping, and delivery capacities. When water use cannot be maintained within the system's capacity, the plan defines procedures and provides guidance for imposing water restrictions. The plan includes phases for management of the City's water supplies through various circumstances, including drought conditions or other catastrophic events that would result in a water shortage.

The extent to which drought restrictions are implemented is primarily based on the flows in the Platte River and water usage. Watering restrictions are implemented through the City's Municipal Code. The various phases of watering restrictions start as voluntary and then increase to mandatory as the severity of the drought increases. Tiered water shortage rates are applied during periods when Water Management Plan restrictions are implemented. The water shortage rates were developed on the basis that customers practicing conservation techniques would see little or no increase in their summer water bills. The water shortage rates begin with the voluntary restrictions and are increased if stricter plan phases are enacted.

The Water Management Plan encourages customers to practice daily conservation, whether or not water restrictions and water shortage rates are applied.

Mayor's Water Conservation Task Force

The City's commitment to water conservation is further demonstrated by the Mayor's Water Conservation Task Force. The Mayor's Water Conservation Task Force is composed of community members appointed by the Mayor. The focus of the Mayor's Water Conservation Task Force is to promote voluntary cooperation to accomplish conservation goals, including:

1. Keep peak day water use within water system's ability to deliver.
2. Encourage participation and support for water conservation practices from business, industry, and the community.
3. Identify and promote adoption of water conserving plant materials and landscape practices.

Subcommittees accomplish the above goals by providing education about the importance of water conservation; improving outdoor and domestic in-home water conservation; improving the efficiency of industrial, commercial, and business water users; and informing customers about water quality.

The Mayor's Water Conservation Task Force promotes water conservation through public information and education. The materials produced by the Mayor's Water Conservation Task Force include public service announcements, educational materials on indoor and outdoor water conservation techniques, and events and contests related to water conservation.

6.2 Potential Water Conservation Practices

In addition to the demand management, tiered rate structure, education, and public information efforts the City has already implemented, other potential water conservation practices may be feasible, including the following:

1. <u>Customer water survey and audit programs</u> – The water survey program includes water use audits for customers and instructs customers on ways to save water in their homes. Auditors perform leak detection tests on meters, plumbing, and irrigation systems.
2. <u>System water audits, leak detection, and leak repair</u> – The goal of a system water audit is to reduce unbilled water in the system. This includes the water that is used for flushing hydrants, fire fighting, water main breaks, leaks, and maintenance activities. The unbilled, or non-revenue, water in the LWS system is already low, at an average of 6.7 percent of the average day demand from 2000 to 2012.

3. Large landscaping conservation programs and incentives – Large landscape programs may include irrigation and landscape audits at sites with a large area (1 acre or more) of irrigated landscape. The programs may also include water budgets and rate incentives for commercial and industrial accounts.

4. Incentive programs for water-efficient fixtures and appliances – Programs may include financial incentives and rebates for retrofit installation of water-efficient fixtures and appliances, including ultra-low-flush toilets, high-efficiency washing machines, and low-flow showerheads.

5. Conservation programs for commercial, industrial, and institutional accounts – Programs may include rebates that customers can apply toward replacing inefficient equipment or implementing new conservation techniques. Other objectives could be to assist customers in identifying retrofitting options or establishing water reduction goals.

7.0 Alternatives Analysis – Supply Options

An alternatives analysis of the short-, mid-, and long-term supply options was developed using a paired matrix approach. This type of analysis helps determine the relative importance of a number of different options. In a paired matrix comparison, each criterion is individually compared against the remaining criteria to determine the relative importance.

To obtain information on the weighting criteria for this evaluation, LWS staff and the LWS stakeholder advisory committee were provided (as separate groups) a survey where they completed a paired matrix evaluation of 10 evaluation criteria. The results of each group's paired matrix analysis were averaged together to determine the overall weighting factor for the subsequent alternatives analysis. The results of the survey are presented in Table 7-1. As shown, the three most important criteria were sustainability, desirable water quality, and the ability to expand in the future.

Table 7-1 Results of Paired Matrix Analysis

Evaluation Criteria	Weighting Factor (%)
Sustainability	15.05
Desirable water quality	14.75
Expandable for future demands	12.40
Increases reliability during drought	11.84
Cost effective	10.70
Optimizes existing infrastructure	9.76
Ease of permitting	8.05
Minimizes environmental impacts	6.34
Minimizes implementation risk	6.10
Implementation Time	5.00

The weighting factors developed from the paired matrix analysis were then used to develop a ranking of the short-, mid-, and long-term supply options. For the short-term options, the new well field in the High Plains Aquifer alternative was not included due to concerns regarding water quality. Additionally, conservation was not considered as a supply option as a part of this evaluation but could be a means of delaying the need for implementation of new sources of supply by reducing demands..

7.1 Summary of Alternatives Analysis

The three alternatives that received the highest scores are 1) the short-term alternative to develop the fourth HCW, 2) the mid-term alternative to expand the City's well field, and 3) the long-term alternative to develop the Missouri River Project. The following sections present recommendations for future water supply development.

7.1.1 Recommended Short-term Option

The alternatives analysis indicated that the construction of the fourth HCW was the preferred short-term alternative. This alternative received the highest score of all alternatives, including the mid-term and long-term alternatives. Therefore, expansion of the existing well field through construction of the fourth HCW is the recommended approach for reducing the short-term deficit between projected water demand and the water supply capacity of the existing infrastructure.

7.1.2 Recommended Mid-Term Option

The expansion of the existing well field received the highest score in the alternatives analysis for mid-term options. Therefore, the expansion of the existing well field is the recommended option.

Based on the analysis previously presented, the construction of a fifth and sixth HCW would increase the summer seasonal pumping capacity of the well field to between 101 and 107 mgd during drought conditions. This pumping capacity would meet projected seasonal demands to approximately 2045. The addition of two new HCWs would increase the maximum instantaneous pumping capacity of the well field to 175 mgd (155 mgd during drought), as shown in Table 4-1. These additional HCWs would address peak day demand conditions during a drought through approximately 2050. The fifth HCW should be operational by 2025 and would address projected increases in water demand to 2035. The sixth HCW should be operational before 2035 and would address projected seasonal demands to approximately 2045.

7.1.3 Recommended Long-term Option

The Missouri River Project will provide the required raw water supply to meet projected demands to 2060 and will increase the reliability of the City's water supply through source diversification.

Future geologic exploration along the Missouri River is required to improve the understanding of both the geologic and water quality conditions in the area. It is generally recommended to locate a HCW as close to the river as possible to maximize induced infiltration for optimal yield and to maximize the percentage of river water to optimize water quality.

Subsurface exploration should be conducted in phases. The first phase should consist of the drilling of soil borings and installation of monitoring wells to collect additional geologic and water quality data in areas that could be developed as a well field. The results of the first phase of work would be used to develop a recommended location for a HCW. Once a location of the development of a HCW is identified, a second phase of detailed aquifer testing should be performed.

8.0 Conclusions

The purpose of this chapter of the Master Plan was to establish the current capacity of LWS's water supply and to identify short-term and long-term alternatives to expand that supply to meet projected water demands through the year 2060. The planning effort included a review of hydrological data to determine the reoccurrence interval of prolonged drought events, a review current well performance and groundwater modeling to determine the seasonal firm capacity of the well field, and an alternatives evaluation to develop recommended short-term and long-term approaches to meet the projected water demands.

The hydrological analysis performed indicated that there is a strong probability that the City will experience at least one drought event of similar magnitude to the 2012 drought during this planning horizon. Therefore, planning to have sufficient water supply to meet projected water supply demands under these conditions is prudent.

8.1 Current Conditions

The analysis of the supply capacity of the existing well field indicated that once construction of the third HCW is complete, the well field will have a maximum instantaneous capacity of between 110 and 130 mgd, depending on streamflow conditions. The summer seasonal capacity of the well field for 60- to 90-day production duration ranges from 75 to 80 mgd when streamflow is less than 1,000 cfs and from 71 to 77 mgd during a streamflow event that correlates to a 100-year reoccurrence interval drought.

8.2 Short-Term Improvements (2014 to 2025)

Even with the completion of the third HCW, a potential supply deficit could occur during prolonged periods of low streamflow as early as 2018, depending on the magnitude and duration of a drought. In 2018, a supply deficit would be anticipated to occur only during extreme drought conditions that correlate to the 50- to 100-year reoccurrence interval event. By 2025, a supply deficit would be anticipated to occur during more frequent drought events such as the 20-year reoccurrence interval event. At this time, there is also a projected supply deficit with the instantaneous and short-term pumping capacity of the well field, where it is projected that the well field may not be able to meet the maximum day demand as early as 2020 during times when the 1-day streamflow is less than the 50- to 100-year reoccurrence interval drought. Expansion of the existing well field through construction of the fourth HCW is recommended to address this potential short term supply deficit. As shown in Table 8-1, it is recommended that the remaining work required to place the fourth HCW into service be completed by 2017.

8.3 Mid-Term Improvements (2026 to 2040)

Assuming the fourth HCW is constructed as recommended, a supply deficit could occur as soon as 2025. To address this mid-term supply deficit, further development of the east bank of the well field is recommended. Two additional (fifth and sixth) HCWs constructed will provide sufficient capacity though 2045.

When both the fifth and sixth HCWs are constructed, the total summer seasonal pumping capacity of the well field is estimated to be between 101 and 107 mgd during drought conditions. This pumping capacity would meet projected seasonal demands to approximately 2045. The addition of the fifth and sixth HCWs would increase the maximum instantaneous pumping capacity of the well field to 175 mgd (155 mgd during drought), and would address peak day demand conditions during a drought through approximately 2050. The fifth HCW should be operational by 2025 and would address projected increases in water demand to 2035. The sixth HCW should be operational before 2035. An additional 48-inch raw water transmission main would be needed to covey water from the fifth and sixth HCWs by 2025.

8.4 Long-Term Improvements (2041 to 2060)

Meeting projected water supply demands beyond 2045 presents an opportunity to diversify the City's water supply sources. Diversification of sources will greatly improve the reliability of the water system during prolonged drought events and will ensure that the City can reliably meet projected water demands through 2060. The recommended long-term supply option is the development of a new well field and water treatment plant.

8.5 Summary of Improvements

The various recommended improvements (and associated costs) for water supply are summarized in Table 8-1. The development of the Missouri River Project will improve the reliability of the City's water supply and also provides the opportunity to develop a regional source of supply.

Table 8-1 Opinion of Probable Cost – Summary of All Improvements

Year	Description	Current Cost Basis[1]	Future Cost Basis – 3% Inflation[2]	Future Cost Basis – 5% Inflation[3]
Immediate Projects				
2016	Rehab Existing Wells	$196,000	$214,000	$227,000
2017-2022	Ongoing Rehab/Replace of Existing Wells	$300,000	$338,000-$391,000	$365,000-$465,000
Short-Term Projects				
2016	Fourth HCW River Crossing/Bank Stabilization	$4,200,000	$4,600,000	$4,900,000
2016	Equip fourth HCW Cassion with Well House, Pumps and Electrical, Roads/Transmission Piping	$6,100,000	$6,700,000	$7,100,000
Mid-Term Projects				
2024	Construct fifth HCW on East Bank (including roads and transmission piping)	$12,600,000	$17,000,000	$22,000,000
2034	Construct sixth HCW on East Bank (including roads and raw water transmission main)	$24,300,000	$45,000,000	$68,000,000
Long-Term Projects				
2016	Collector Well Investigation – 2 Sites for Missouri River Project	$550,000	$601,000	$637,000
2018	Missouri River Project – Well Field Property Purchase	$2,410,000	$2,800,000	$3,100,000
2040	Missouri River Project	$499,500,000	$1,200,000,000	$1,900,000,000

Notes:
1. *Engineering and Contingency estimates are included in each item at a value of Contingency 25% and Engineering 15% of the item cost.*
2. *Inflated to projected year dollars at 3% per year inflation rate through 2017.*
3. *Inflated to projected year dollars at 5% per year inflation rate for years beyond 2017.*

Lincoln Water System Facilities Master Plan

Chapter 4 - Water Treatment

May 2014

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Abbreviations and Acronyms

2002 Master Plan	2002 Facilities Master Plan
°C	degrees Celsius
CCR	consumer confidence report
cfu	colony forming unit
City	City of Lincoln
CT	contact time
cVOC	carcinogenic volatile organic compound
DBP	disinfection byproducts
DBPR	Disinfectant/Disinfection Byproduct Rule
EPA	U.S. Environmental Protection Agency
FY	fiscal year
GAC	granular activated carbon
gpm	gallons per minute
HAA5	haloacetic acids
HCW	horizontal collector well
HDR	HDR Engineering, Inc.
IESWTR	Interim Enhanced Surface Water Treatment Rule
KWH	kilowatt hour
lb/d	pounds per day
LCR	Lead and Copper Rule
LRAA	locational running annual average
LT2ESWTR	Long-Term 2 Enhanced Surface Water Treatment Rule
LWS	Lincoln Water System
Master Plan	2013 Facilities Master Plan
MCL	Maximum Contaminant Level
MCLG	Maximum Contaminant Level Goal
MG	million gallons
MGD	million gallons per day
mg/L	milligrams per liter

µg/l	micrograms per liter
NDMA	Nitrosodimethylamine
NOM	natural organic matter
NPDES	National Pollutant Discharge Elimination System
NTU	Nephelometric Turbidity Units
OPPD	Omaha Public Power District
PLSLR	Partial Lead Service Line Replacement
RTCR	Revised Total Coliform Rule
SAB	Science Advisory Board
SDS	Simulated Distribution System
SDWA	Safe Drinking Water Act
SWTR	Surface Water Treatment Rule
TCR	Total Coliform Rule
TOC	total organic carbon
TTHM	total trihalomethanes
UCMR3	Third Unregulated Contaminant Monitoring Rule
UON	unless otherwise noted
UV	ultraviolet
WHO	World Health Organization

1.0 Introduction

Chapter 4 - Water Treatment provides an assessment of the existing treatment facilities and defines the improvements required to meet future regulatory requirements and capacity needs of the system.

2.0 Overview of Existing Facilities

The Lincoln Water System's (LWS's) Platte River Water Treatment Facility consists of two treatment plants located on a single site. The East Plant has a design capacity of 60 million gallons per day (MGD) and treats water to surface water treatment standards. The West Plant is a 60 MGD groundwater treatment plant.

2.1 East Plant

The East Plant was constructed in 1994 and has a maximum production capacity of 60 MGD. The plant treats water from two 17.5-MGD horizontal collector wells (HCWs) in the City of Lincoln's (City) well field. The source water from one of the existing HCWs is classified as groundwater under the direct influence of surface water. A third HCW with a similar expected capacity is currently under construction. The caisson and laterals for a fourth HCW are also under construction; the well is not currently being equipped with piping, pumps, or well house.

The East Plant treatment processes are shown in the process flow diagram in Figure 2-1 and summarized below:

- Ozonation for iron and manganese oxidation and primary disinfection
- Addition of filter aid polymer
- Gravity filtration
- Primary disinfection with free chlorine through filters, clearwells, 84-inch pipeline
- Secondary disinfection with chloramines though storage reservoir
- Fluoridation
- Finished water pumping through south transmission pumping station

The facility has been designed to provide for four future 30-MGD treatment trains, future chemical feed equipment, and the expansion of the reservoir and transmission pumping station. Primary power for the plant is provided by two redundant power services from Omaha Public Power District (OPPD). The plant is equipped with transfer switches to allow mobile diesel generators to provide backup power for the entire facility.

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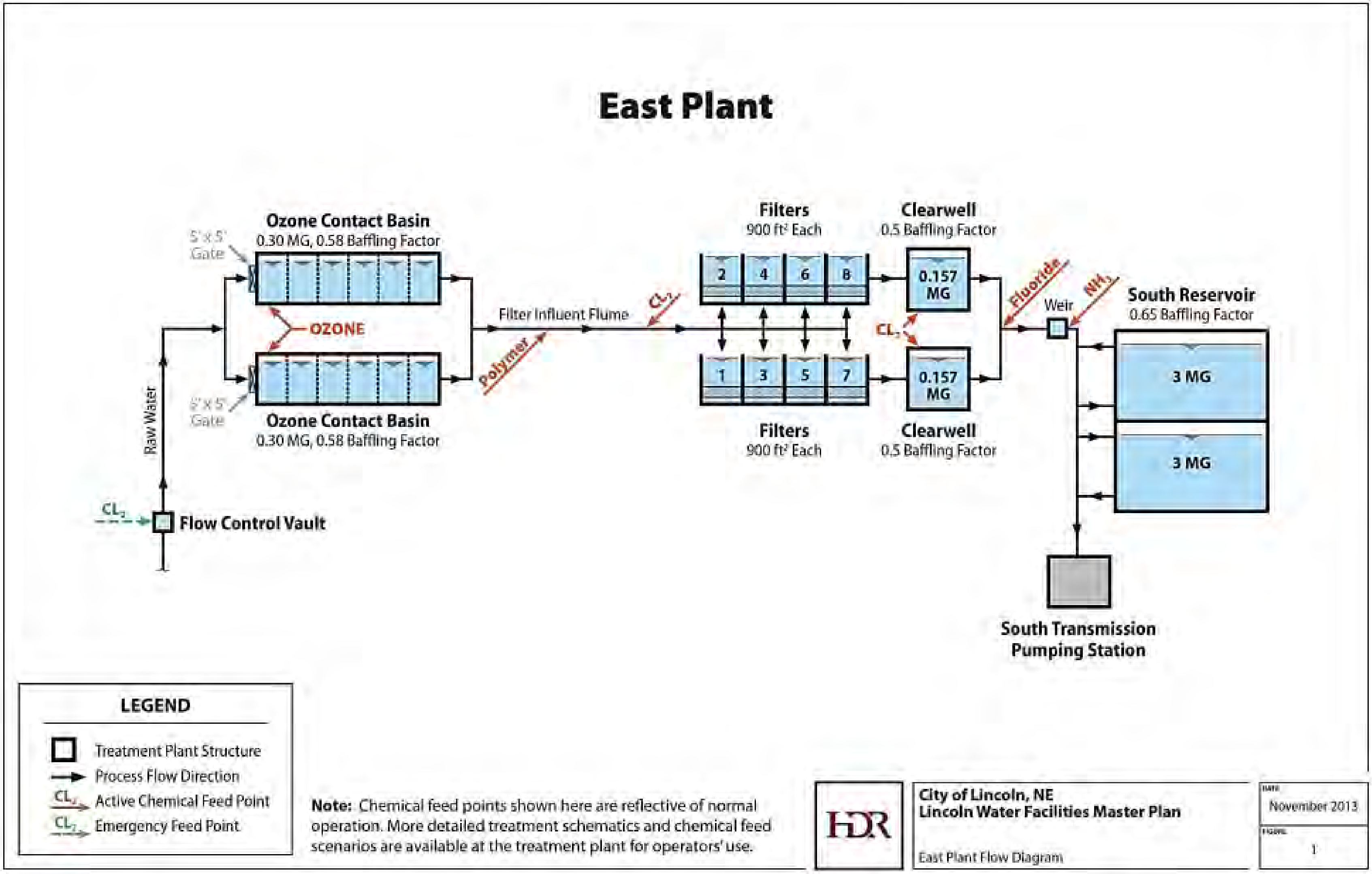

Figure 2-1 East Plant Process Flow Diagram

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2.2 West Plant

The West Plant has a treatment capacity of 60 MGD. It treats water from 40 vertical wells in the City's well field with capacities ranging from 500 to 3,000 gallons per minute (gpm). The water from all of the vertical wells is classified as groundwater. The West Plant was constructed in 1935 and underwent major expansions in 1948, 1954, and 1956.

The West Plant treatment processes are shown in the process flow diagram in Figure 2-2 and summarized below:

- Aeration for oxidation of iron
- Primary disinfection with free chlorine through detention basins (Chlorination also provides oxidation of manganese through detention basins.)
- Gravity filtration
- Secondary disinfection with chloramines though clearwells and reservoir
- Fluoridation
- Finished water pumping through north transmission pumping station

As with the East Plant, primary power for the plant is provided by two power services from OPPD. The plant is equipped with transfer switches to allow mobile diesel generators to provide backup power for the entire facility.

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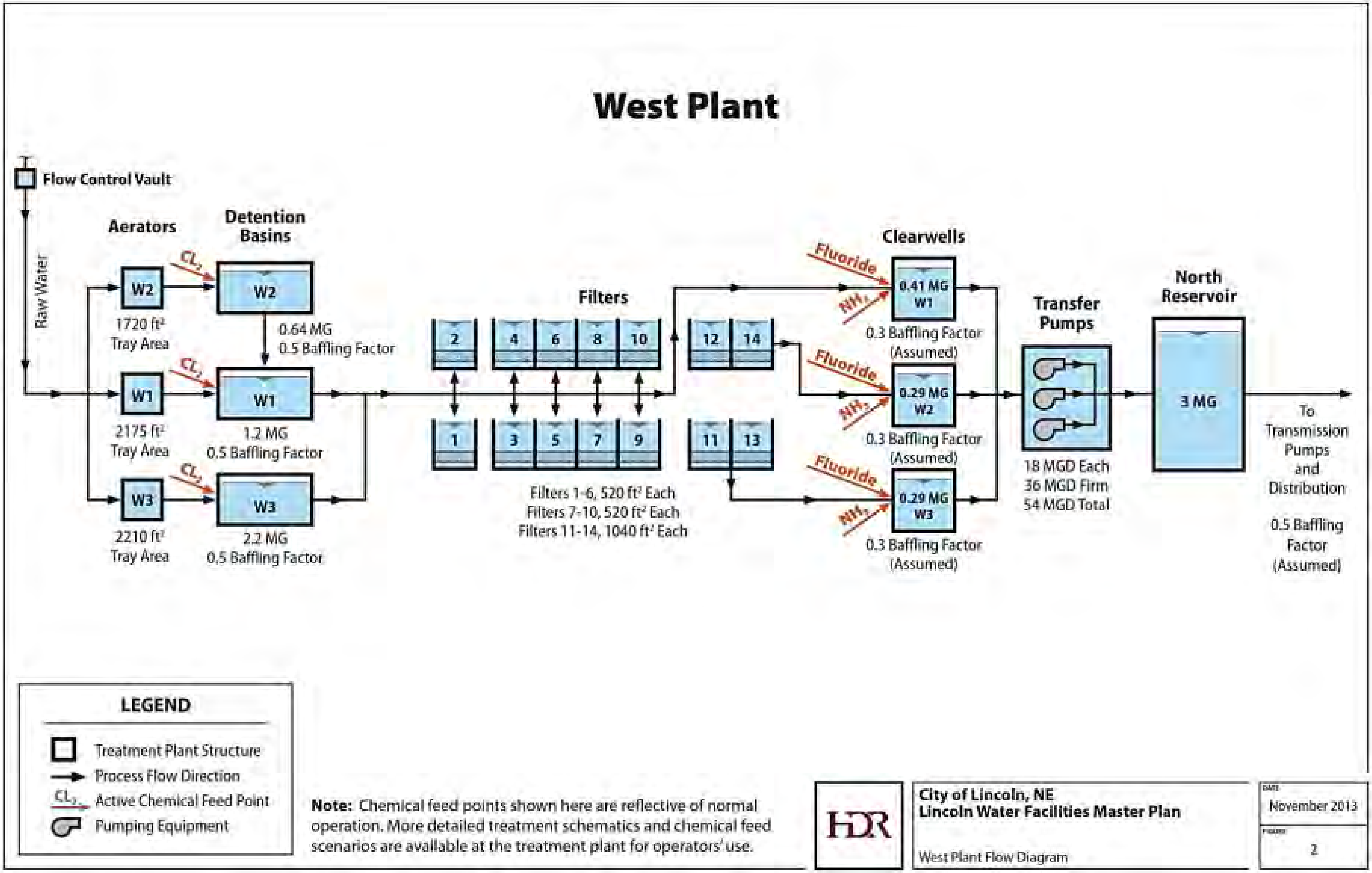

Figure 2-2 West Plant Process Flow Diagram

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2.3 Energy Use

Historical energy use at the Platte River Water Treatment Facility from fiscal years (FY) 1994/95 through 2012/13 is summarized in Table 2-1 below.

Table 2-1 Historical Energy Use - Fiscal Years 1994/95 Through 2012/13

Fiscal Year	KWHs	Total Electrical Expense	Cost/KWH	Transmission Water Pumped, MG	KWHs/MG
94/95	12,222,000	$579,240	$0.0473	12,498	1,085.7
95/96	13,066,200	$497,430	$0.0380	12,068	1,134.7
96/97	13,834,800	$527,741	$0.0381	12,868	1,179.1
97/98	14,301,000	$536,493	$0.0375	12,452	1,218.9
98/99	14,187,600	$559,777	$0.0395	12,366	1,232.4
99/00	16,619,400	$563,716	$0.0339	15,330	1,159.8
00/01	17,933,855	$607,498	$0.0339	14,365	1,272.7
01/02	17,736,544	$611,508	$0.0345	14,620	1,227.4
02/03	17,388,000	$593,570	$0.0341	13,930	1,248.2
03/04	14,817,999	$486,146	$0.0328	13,804	1,080.7
04/05	14,741,967	$476,641	$0.0323	14,459	1,070.3
05/06	15,409,798	$497,799	$0.0323	14,870	1,085.4
06/07	13,754,897	$624,367	$0.0454	13,422	1,030.7
07/08	12,922,744	$511,091	$0.0395	12,526	1,032.7
08/09	13,435,153	$550,277	$0. 410	12,693	1,062.5
09/10	12,416,349	$535,003	$0.0431	11,622	1,068.3
10/11	13,044,422	$616,022	$0.0472	12,600	1,042.5
11/12	13,723,854	$685,406	$0.0499	14,005	985.6
12/13	12,830,832	$699,093	$0.055	13,218	972.6

The trends of the summarized parameters are shown in Figures 2-3 through 2-7.

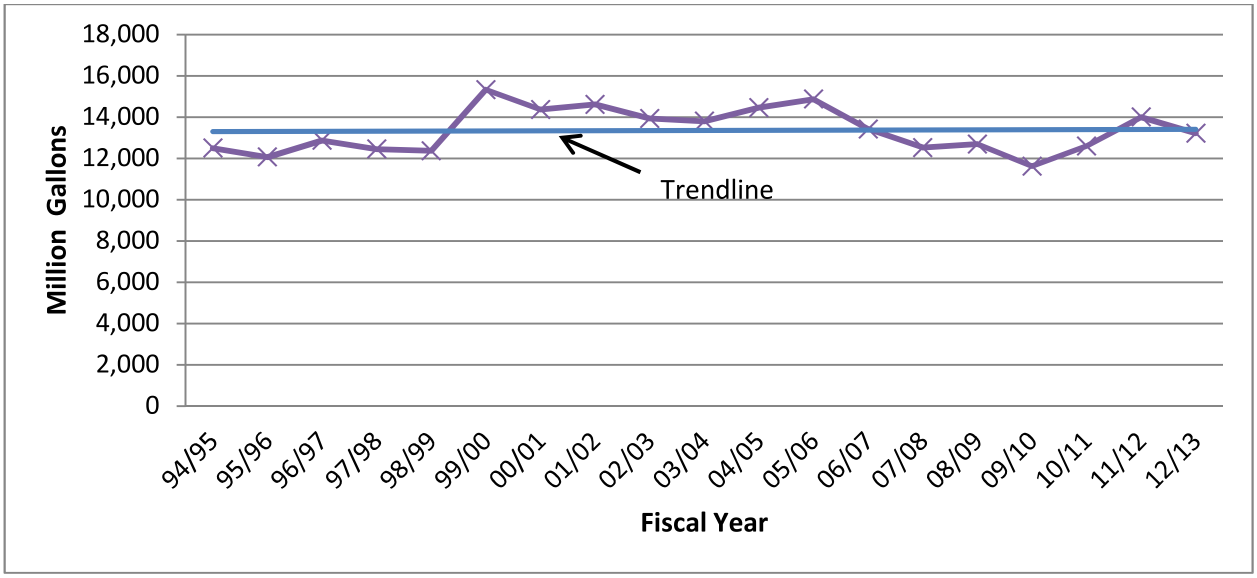

Figure 2-3 Volume of Water Pumped by Transmission Pumps per Fiscal Year

The amount of water pumped by high service (transmission) pumps has fluctuated between about 11,620 MG and 15,330 MG.

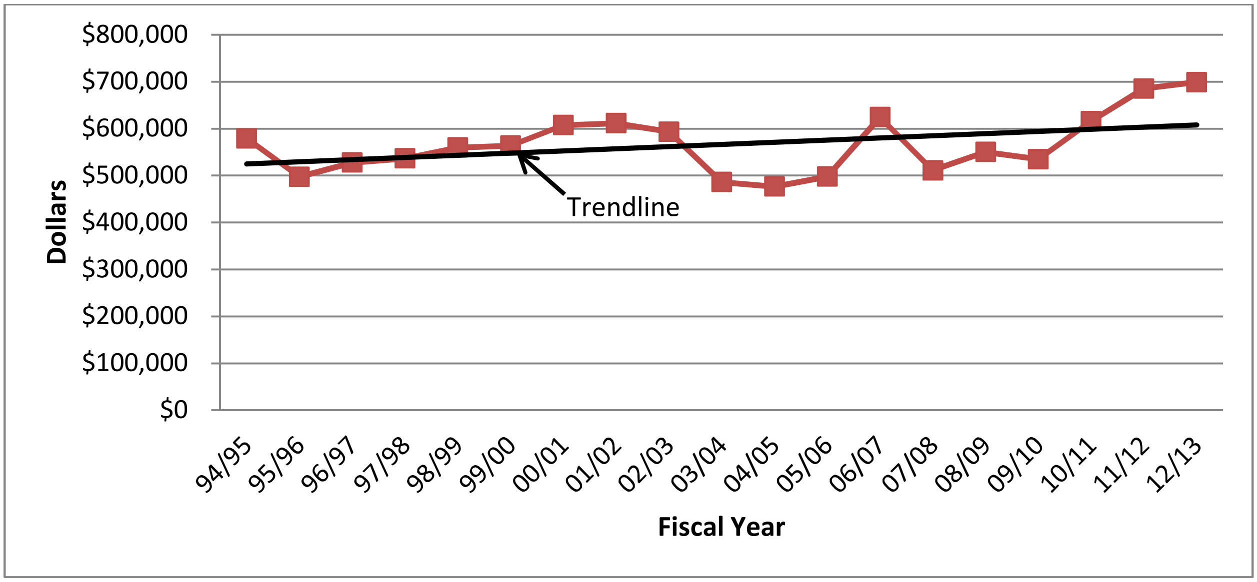

Figure 2-4 Total Electrical Expenses per Fiscal Year

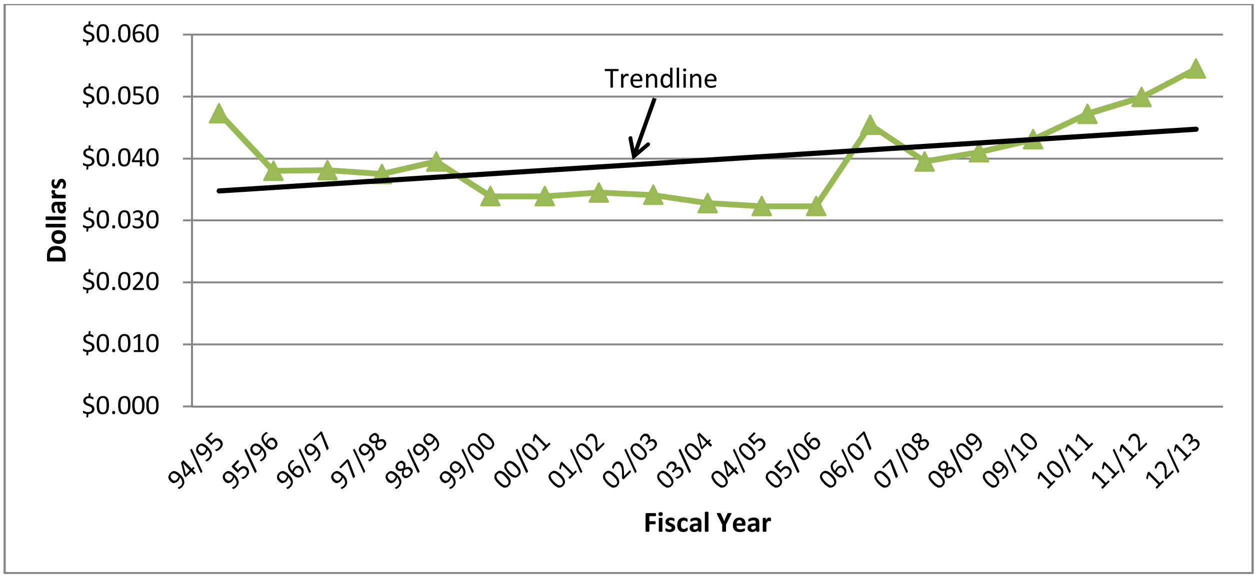

Figure 2-5 Cost/KWH per Fiscal Year

The total electrical expenses have ranged from a low of about $476,600 in FY 04/05 to a high of about $699,000 in FY 12/13. The cost per KWH was lowest in FY 04/05 through 05/06 when it was $0.0323/KWH. The highest historical cost per KWH was in FY 12/13 at a cost of $0.055/KWH. Both parameters exhibit the same general increasing trend.

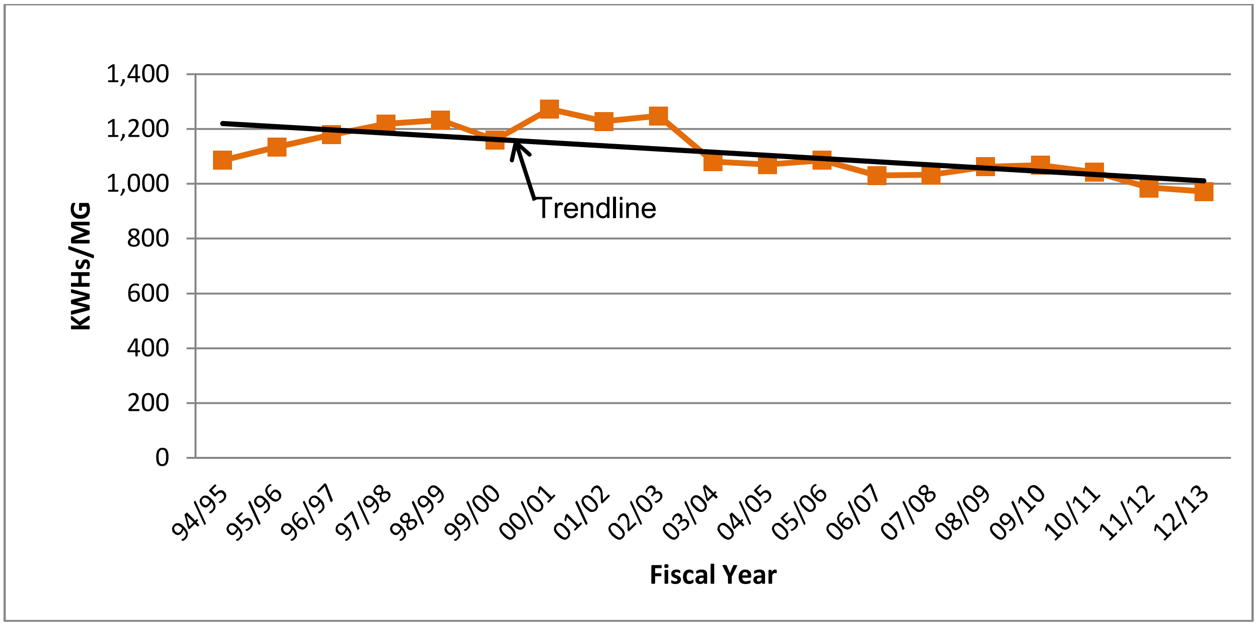

Figure 2-6 Energy Usage in KWHs/Million Gallons per Fiscal Year

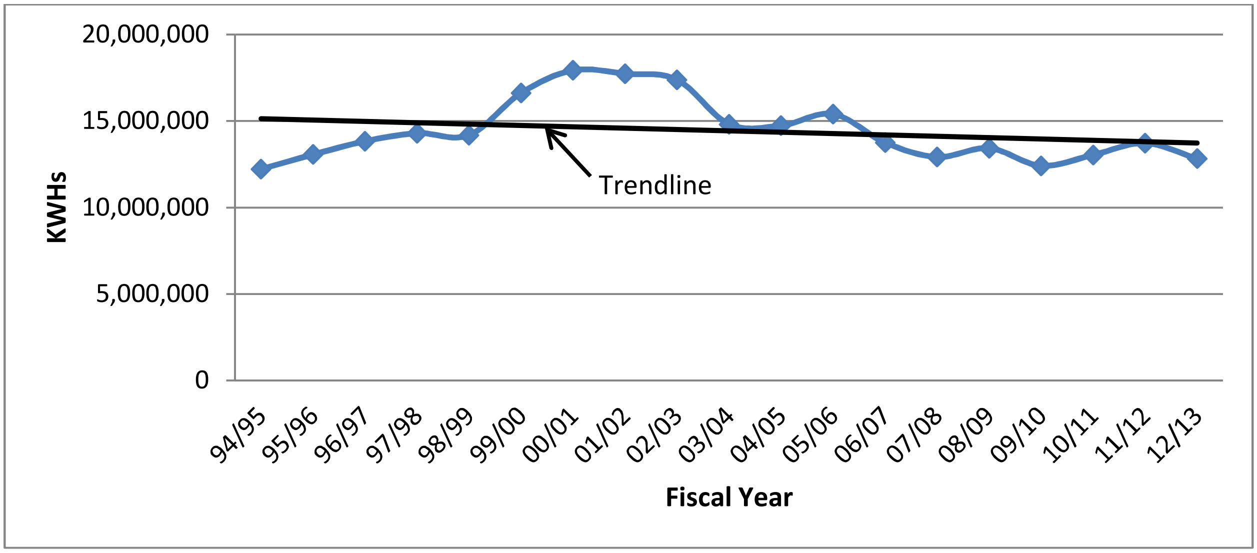

Figure 2-7 Energy Usage in Total KWHs Per Fiscal Year

The energy usage shown in KWHs/MG and total KWHs per year reveals the same general downward trend over the historical time period. The KWH/MG has consistently fallen each year from about 1,080 KWH/MG in FY 03/04 to 973 KWH/MG in FY 12/13. The downward trends of these parameters show the effectiveness of LWS's improvements in efficiencies and demand management.

High service pumping is a primary area of energy use at the treatment facilities and within the distribution system. The power used at the Platte River Water Treatment Facility is supplied by OPPD. LWS has developed specific combinations of pumps to be used in different scenarios to control energy uses and power charges. The pump combinations are dictated by parameters such as:

- Pumping season and Off-Peak or On-Peak hours
- Flow rate and head condition
- Pumping efficiency to minimize energy consumption and electrical demand charges

LWS has the ability to use diesel pumps to keep demand levels below certain values at specific plant production rates. The diesel pumps are generally more expensive to operate per unit of water pumped than the electric pumps.

The electric rates that apply to the Platte River Water Treatment Facility include higher energy charges during the dates of June 1 through September 30. The OPPD rate schedule also includes a Time of Use Rider, which considers demands during the hours of 12:00 noon and

10:00 p.m., Monday through Friday, from June 1 through September 15 as On-Peak demands. The On-Peak demands that are established impact the billing rates for the next 11 months. LWS may request a special waiver from On-Peak demand billing calculation for a demand that will be higher than a set base-level for a specified period of time. If OPPD determines that the energy and capacity are available, LWS may decide to purchase the energy and capacity at the price established by OPPD to avoid having the On-Peak billing demand increased.

LWS has practiced energy demand management effectively. In the past 12 years since Fiscal Year (FY) 00/01, LWS has saved an average of $279,978 per year, or $3,359,736 total, due to their efforts in energy demand management. Demand management efforts have consisted of pumping during Off-Peak periods when possible, cost/KWH savings per OPPD calculations, load sharing reductions, and completion of the 60-inch finished water pipeline, which allows LWS to pump directly to the distribution system.

It is recommended that LWS continue its proactive approach to demand management and conservation to ensure continued energy savings, especially as the overall trend of energy costs rises. Additional energy savings efforts could include converting more pumps to variable speed units. Alternative energy technologies should be considered when equipment is replaced or added. Examples of alternative technology implementations could include the conversion of base load pumping units to natural gas, or adding a natural gas generator.

3.0 Water Quality and Regulatory Requirements

3.1 Summary of Existing Raw and Finished Water Quality

To properly evaluate the existing treatment facilities, review of raw water and finished water quality parameters is necessary to determine the effectiveness of the existing treatment processes and provide recommendations of future treatment improvements. As mentioned previously, the Platte River Water Treatment Facility has two separate treatment plants: the East Plant and the West Plant. The East Plant treats a combination of groundwater and groundwater under the direct influence of surface water. The West Plant treats only groundwater. Due to the difference in source water, raw water quality at each plant and the required treatment differ. Therefore, the water quality data are presented for each plant separately.

Data from the past 3 years were provided by LWS. This time period was selected based on the varying amount of rainfall experienced, from above average rainfall to very dry periods. The amount of rainfall directly affects the aquifer from which LWS draws water and impacts the concentration of various constituents in the water. Tables 3-1, 3-2, and 3-3 include a summary of data from 3 years of sampling and reporting, including data from years 2010, 2011, and 2012. LWS has consistently been in compliance with all drinking water standards.

Table 3-1 Raw Water Quality[1]

Parameter	East Raw Water			West Raw Water		
	Average	Min	Max	Average	Min	Max
	(mg/L UON)[3]					
Acetochlor (µg/L)	0.09	0.00	0.85	0.01	ND[2]	0.31
Alachlor (µg/L)	ND[2]	ND[2]	ND[2]	0.01	ND[2]	0.27
Atrazine (µg/L)	0.78	0.00	4.42	0.29	ND[2]	0.52
Bromide	0.78	0.043	0.188	0.073	0.18	0.103
Coliform, Total (MPN/100ml)	0	0	0	0	0	0
Cryptosporidium (#/L)	0	0	0	0	0	0
Cyanazine (µg/L)	ND[2]	ND[2]	ND[2]	ND[2]	ND[2]	ND[2]
Desethylatrazine (µg/L)	0.14	ND[2]	0.59	0.04	ND[2]	0.20
Desisopropylatrazine (µg/L)	0.01	ND[2]	0.19	0.00	ND[2]	ND[2]
E coli (P/A)	0	0	0	0	0	0
Fluoride	0.396	0.131	0.589	0.426	0.338	0.488
Heterotrophic Plate Count (HPC) (cfu/100ml)	158	3	>999	145	0	>999
Iron	0.004	ND[2]	0.032	0.010	ND[2]	0.109
Manganese, Total	0.038	0.003	0.336	0.057	0.001	0.255
Metolachlor (µg/L)	0.32	ND[2]	1.04	0.20	ND[2]	0.34
Nitrate (as N)	1.24	0.66	2.581	0.33	0.049	0.88
Nitrite (as N)	0.032	0.000	0.354	0.015	0.000	0.451
pH (su)	7.81	7.03	8.07	7.49	7.23	7.95
Simazine (µg/L)	ND[2]	ND[2]	ND[2]	ND[2]	ND[2]	ND[2]
Temperature (°C)	17.7	8.7	27.1	18.0	11.7	28.8
Total Organic Carbon (TOC)	2.94	1.56	5.97	2.25	1.70	3.64
Turbidity (NTU)	0.11	0.05	ND[2]	0.09	0.04	0.38

Notes:

1. *Raw Water samples are collected from sample taps located in the operators' mini-lab. Raw water sources are tapped in the respective raw water mains prior to any treatment.*
2. *ND – Not Detected*
3. *UON – Unless Otherwise Noted*

(Shaded Items are not included in the primary or secondary drinking water regulations.)

Table 3-2 East Plant Finished Water Quality

Parameter	East Finished Water[1]				
	Average	Min	Max	MCL	MCLG
	(µg/L UON)				
Arsenic, Total	8.00	6.40	8.90	10	0
Atrazine	0.14	0.00	0.27	3	3
Barium (mg/L)	0.127	0.127	0.127	2	2
Bromate	0.92	ND[6]	4.50	10	0
Bromodichloromethane	9.97	4.03	15.90	[5]	0
Bromoform	0.54	ND[6]	1.08	[5]	0
Chloroethane	0.39	ND[6]	0.78		
Chloroform	8.93	2.46	15.40	[5]	70
Chloromethane	1.06	ND[6]	2.11		
Chromium	6.24	6.24	6.24	100	100
Dibromochloromethane	6.25	4.80	7.69	[5]	60
Fluoride (mg/L)	0.797	0.797	0.797	4[2]	4
Manganese (mg/L)	0.003	ND[6]	0.018	0.05[4]	
Metolachlor	0.05	ND[6]	0.10		
Nickel	2.07	1.63	2.51		
Nitrate+Nitrite-N (mg/L)	1.143	0.808	1.560	10[3]	
Selenium	ND[6]	ND[6]	ND[6]	50	50
Sulfate (mg/L)	78.80	73.90	83.70	250[4]	
Total Organic Carbon (TOC) (mg/L)	2.95	2.10	4.30		

Notes:

1. *East Plant Finished Water sampling point is POE 001*
2. *Fluoride has a secondary MCL of 2 mg/L*
3. *Nitrate as N and Nitrite as N have individual MCL/MCLG of 10/10 mg/L and 1/1 mg/L respectively*
4. *On the Secondary Drinking Water Regulation list*
5. *Included in MCL for total trihalomethanes (TTHM)*
6. *ND – Not Detected*

(Shaded items are not on the Primary or Secondary Drinking Water Regulation lists.)

Table 3-3 West Plant Finished Water Quality

Parameter	West Finished Water[1]				
	Average	Min	Max	MCL	MCLG
	(µg/L UON)				
Arsenic, Total	6.97	6.28	7.52	10	0
Atrazine	0.09	ND[6]	0.17	3	3
Barium (mg/L)	0.127	0.127	0.127	2	2
Bromodichloromethane	7.83	7.18	8.48	[5]	0
Bromoform	0.26	ND[6]	0.51	[5]	0
Chloroform	5.97	5.04	6.89	[5]	70
Chromium	6.50	6.50	6.50	100	100
Copper (Distribution System) (mg/L)	0.354	0.020	0.964	1.3	1.3
Dibromochloromethane	5.26	5.04	5.47	[5]	60
Fluoride (mg/L)	0.860	0.860	0.860	4[2]	4
HAA5 (Distribution System)	9.59	ND[6]	22.90	60	
Lead (Distribution System) (mg/L)	0.0017	ND[6]	0.0147	0.015	0
Manganese(mg/L)	0.003	ND[6]	0.015	0.05[4]	
Nickel	1.57	1.57	1.57		
Nitrate+Nitrite-N (mg/L)	0.614	0.382	0.825	10[3]	
Selenium	5.27	5.27	5.27	50	50
Sulfate (mg/L)	73.90	73.90	73.90	250[4]	
TTHM (Distribution System)	24.86	12.40	40.70	80	

Notes:
1. *West Plant Finished Water sampling point is POE 003*
2. *Fluoride has a secondary MCL of 2 mg/L*
3. *Nitrate as N and Nitrite as N have individual MCL/MCLG of 10/10 mg/L and 1/1 mg/L respectively*
4. *On the Secondary Drinking Water Regulation list*
5. *Included in MCL for total trihalomethanes (TTHM)*
6. *ND – Not Detected*

(Shaded items are not on the Primary or Secondary Drinking Water Regulation lists.)

LWS has been in compliance with all existing drinking water regulations with the East Plant, the West Plant, and the distribution system. Several water quality parameters and regulations have particular significance for LWS, both today and potentially in the future. These specific water quality parameters and regulations are discussed in the following sections.

3.1.1 Manganese

One of the primary functions of both treatment plants is the removal of manganese by oxidation and filtration. Oxidation of manganese by ozone and chlorine is used in the East Plant and West Plant, respectively. Raw manganese concentrations have averaged 38 and 57 µg/L over the past 3 years in the East Plant and West Plant, respectively. However, the West Plant raw manganese concentrations have been increasing in the past 2 years, possibly due to extended drought conditions and lowered aquifer levels From 2005 through 2011, the average manganese concentration for West Plant raw water was 47 µg/L. In 2012, it was nearly 80 µg/L and in 2013, it was nearly 90 µg/L. Finished manganese concentrations have averaged 2.7 and 2.5 µg/L over the past 3 years in the East Plant and West Plant, respectively. The Secondary Maximum Contaminant Level (MCL) for manganese is 50 µg/L, and the recommended target for keeping the distribution system clean of manganese precipitate is 10 µg/L. East Plant and West Plant raw and finished total manganese concentrations are shown in Figures 3-1 and 3-2, respectively.

LWS has a manganese treatment goal of less than 10 µg/L in the finished water. The East Plant has performed with 99.0 percent of daily finished water samples less than 10 µg/L, only two samples greater than 10 µg/L, and a maximum of 18.5 µg/L in the past 3 years. The West Plant has performed with 99.7 percent of daily finished water samples less than 10 µg/L, only five samples greater than 10 µg/L, and a maximum of 14.8 µg/L in the past 3 years. The unusually high sample values may have resulted from errors in sampling, data entry, or analysis. Both plants perform very well in removal of manganese to very low levels.

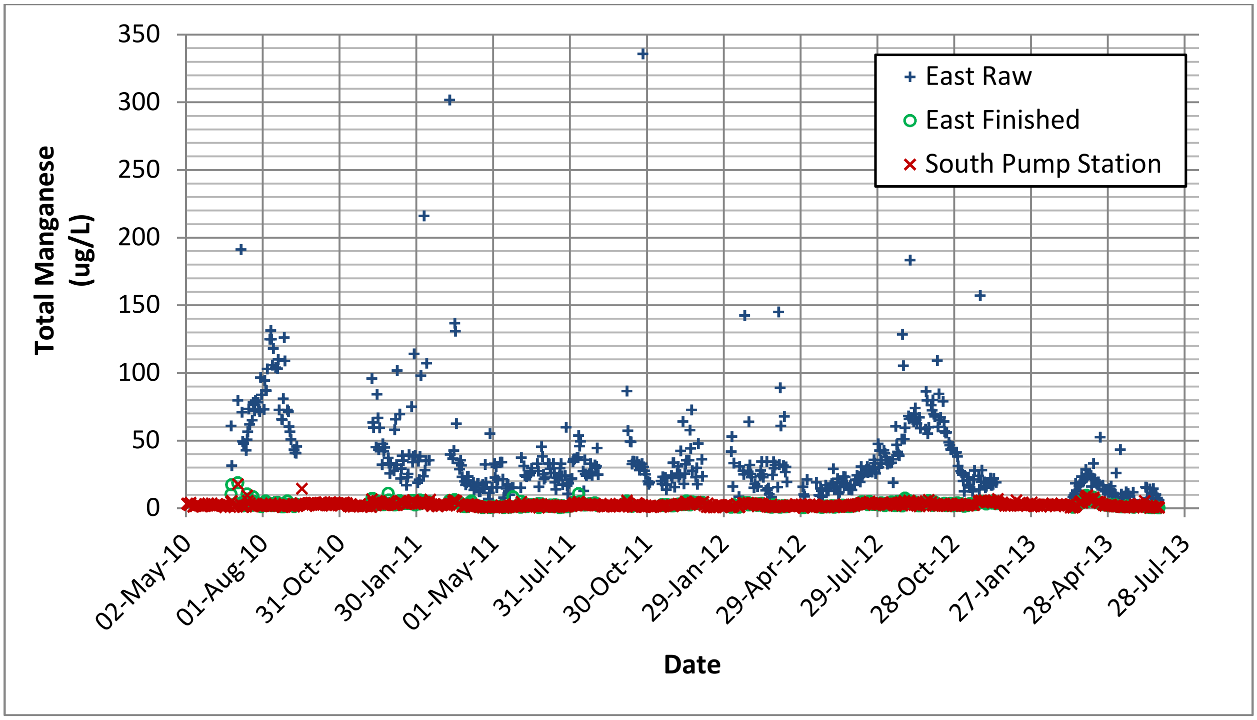

Figure 3-1 East Plant Total Manganese

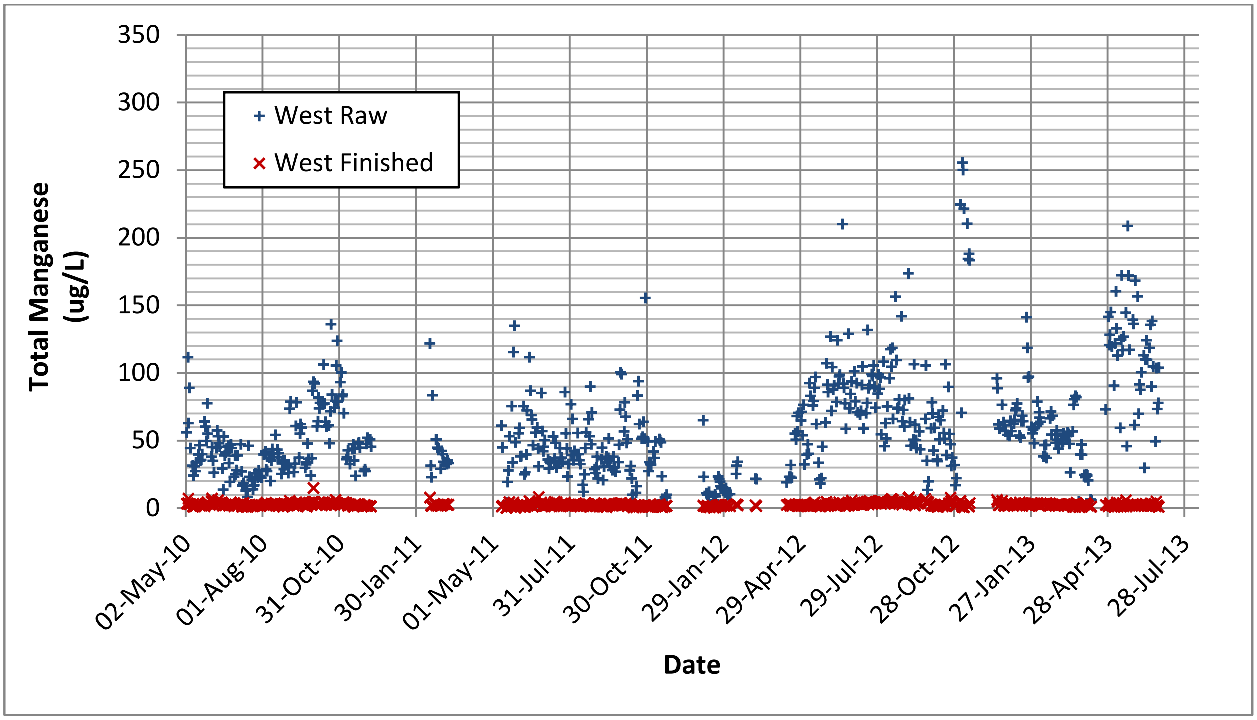

Figure 3-2 West Plant Total Manganese

3.1.2 Turbidity

The East Plant source is classified as groundwater under the direct influence of surface water due to the HCWs' contribution, so the filtration process is held to a higher standard of performance to meet the Long-Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR). The West Plant source currently is classified as a true groundwater with all vertical wells, so the filtration process does not have to meet these strict performance requirements by regulation. The performance standards of the LT2ESWTR are summarized as follows:

Combined filter effluent turbidity must:

- Be less than or equal to 0.3 NTU in at least 95 percent of measurements each month
- At no time exceed 1 NTU

Individual filter effluent turbidity for each filter cannot be:

- Greater than 0.5 NTU in two consecutive measurements taken 15 minutes apart at the end of the first 4 hours of continuous filter operation after the filter has been backwashed or otherwise taken offline
- Greater than or equal to 1.0 NTU in two consecutive measurements taken 15 minutes apart at any time

Violation of the individual filter effluent turbidity performance standards triggers filter profiling and reporting, filter self-assessment and reporting, or comprehensive performance evaluation and reporting. East Plant and West Plant raw and finished turbidities are shown in Figures 3-3 and 3-4, respectively.

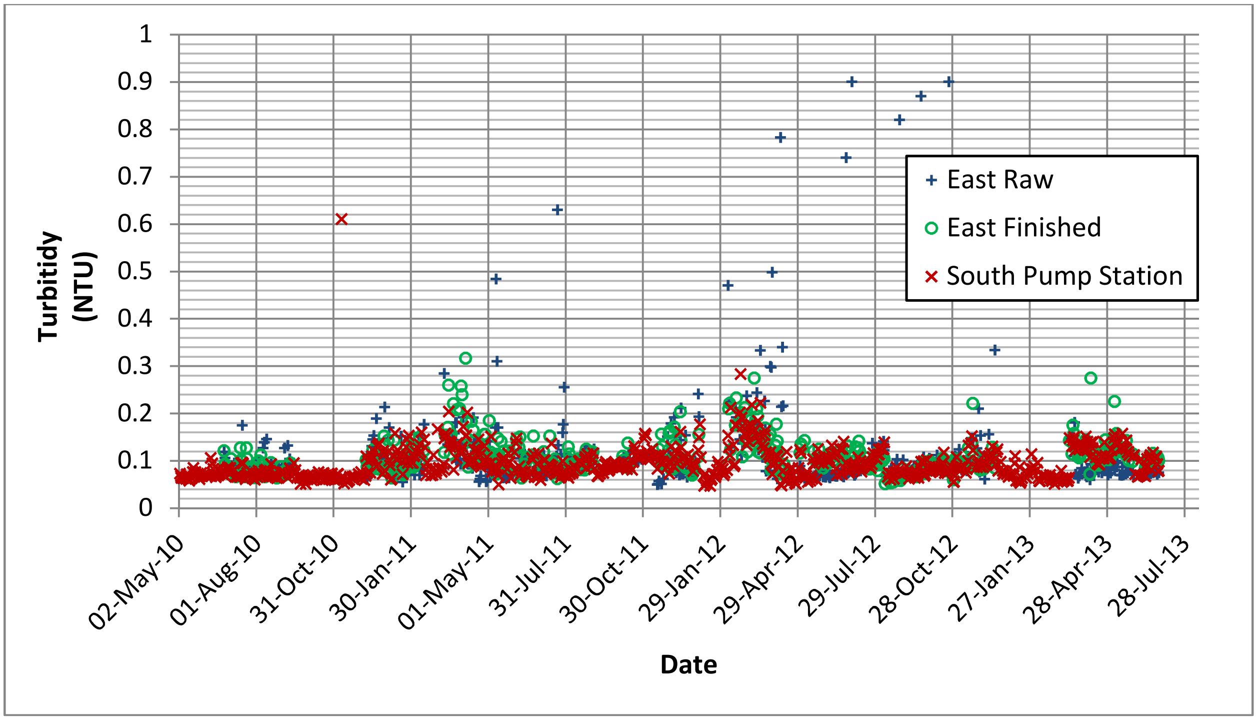

Figure 3-3 East Plant Turbidity

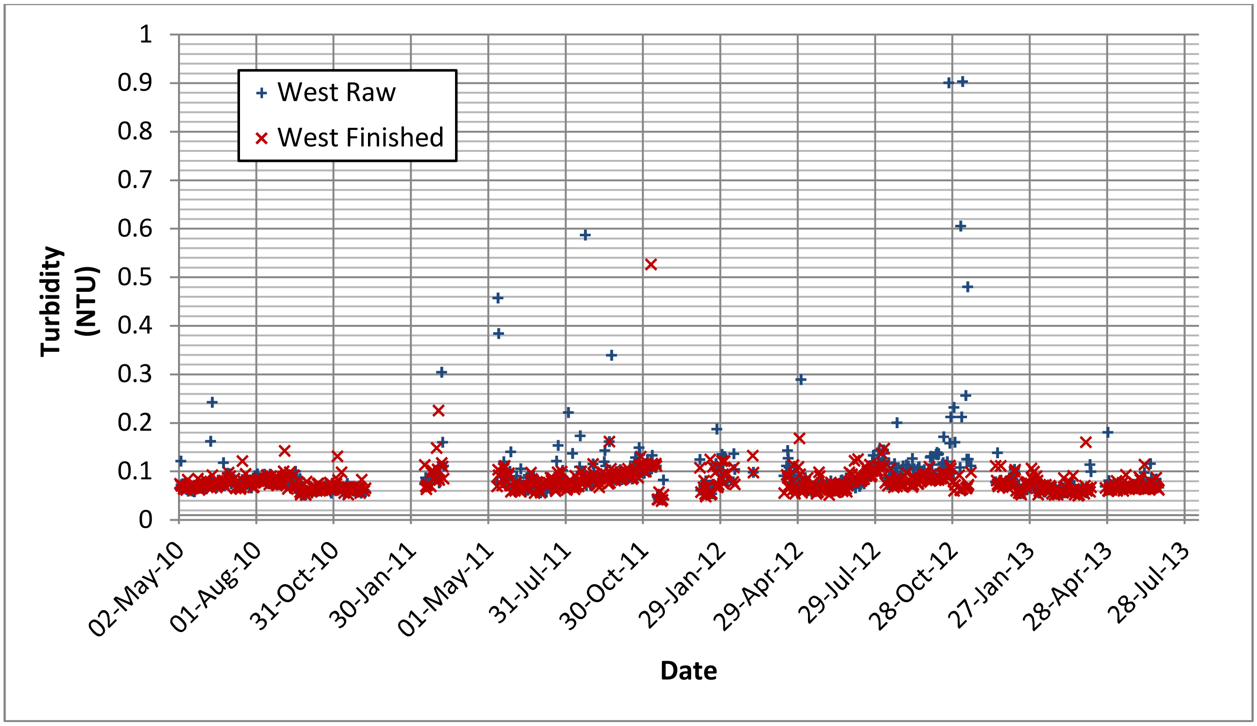

Figure 3-4 West Plant Turbidity

3.1.3 TOC

Natural organic matter (NOM) is measured by the surrogate parameter total organic carbon (TOC). NOM reacts with chlorine, ozone, and other disinfectant/oxidant chemicals to form disinfection byproducts (DBPs), some of which are currently regulated and others that may be regulated in the future. The higher the TOC, the higher the DBPs with the same disinfectant chemical and dose. This results in more challenging compliance with DBP regulations.

East Plant and West Plant raw and finished TOC are indicated graphically in Figures 3-5 and 3-6, respectively. LWS is on reduced quarterly monitoring for TOC. Raw TOC concentrations have averaged 2.94 and 2.25 mg/L over the past 3 years in the East Plant and West Plant, respectively. Maximum raw TOC concentrations of 5.97 and 3.64 mg/L have been indicated for the East Plant and West Plant, respectively. The East Plant TOC has exhibited higher values and more variability due to the contribution of the HCWs.

If HCW water were to be treated in the West Plant in the future, higher average and maximum TOC values can be expected. The higher NOM of the HCW water versus the vertical well water would increase DBP concentrations with the long free chlorine CT of the detention basins in the West Plant. However, if the ratio of HCW water to vertical well water is kept relatively low, then the resulting DBP increase could be small and remain below the MCLs. Simulated Distribution System (SDS) total trihalomethanes (TTHM) and haloacetic acids (HAA5) testing should be conducted using varying ratios of HCW water to vertical well water to determine what amount of HCW water could be used while safely maintaining compliance with the Stage 2 Disinfectant/Disinfection Byproduct Rule (DBPR).

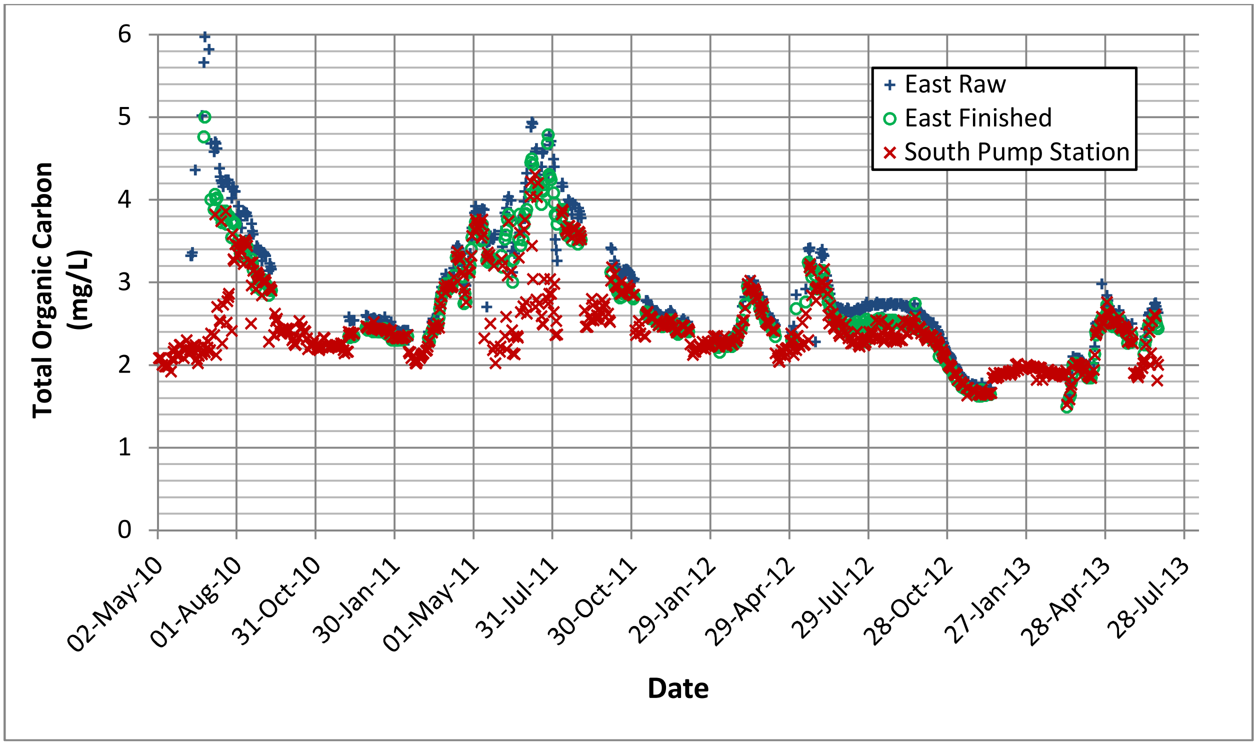

Figure 3-5 East Plant TOC

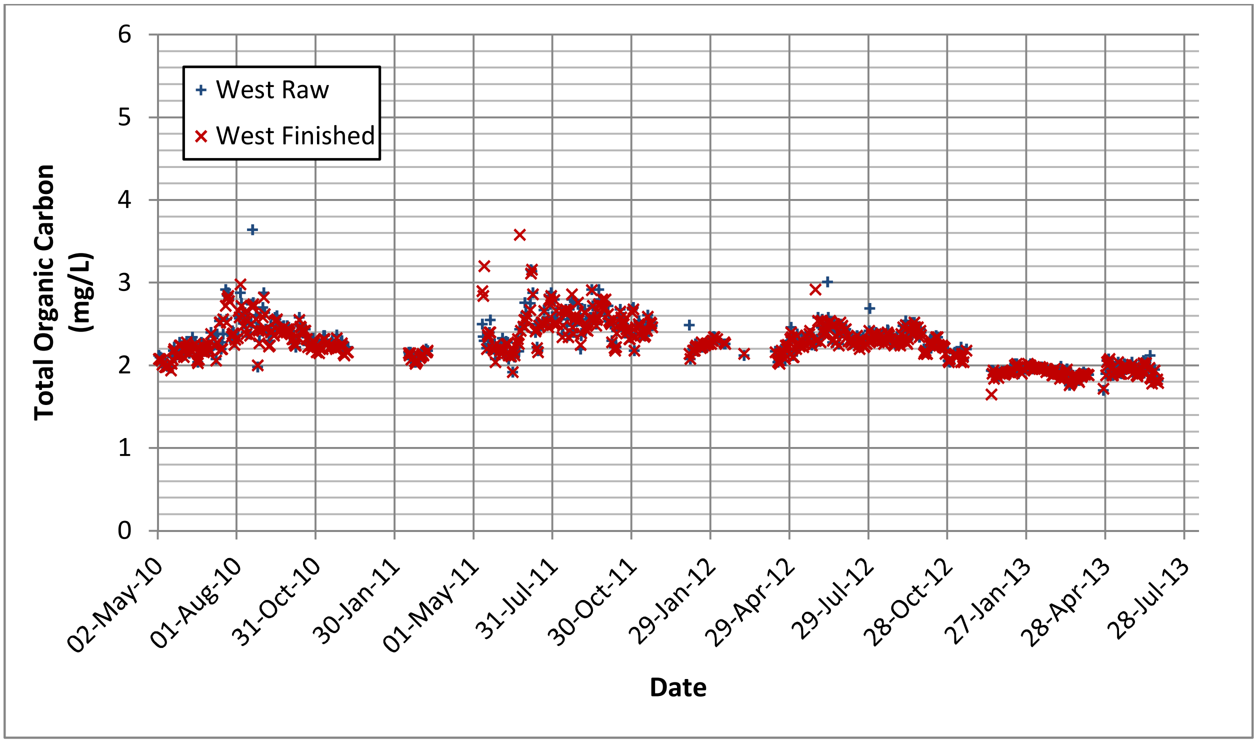

Figure 3-6 West Plant TOC

3.1.4 TTHM and HAA5

TTHM and HAA5 are two groups of DBPs currently regulated under the Stage 2 DBPR. TTHM and HAA5 are formed by reaction of NOM primarily with free chlorine and to a much lesser extent with chloramines in the treatment plants and throughout the distribution system. The majority of TTHM and HAA5 concentrations are produced in the treatment plants during CT with free chlorine. Once ammonia is added and chloramines are formed, the production of TTHM and HAA5 is greatly reduced through the remainder of the treatment plants and throughout the extensive transmission and distribution system.

The East Plant uses ozone for oxidation of manganese and initial disinfection, which does not form TTHM or HAA5. Additional disinfection is provided by free chlorine contact through the filter clearwells and then chloramines throughout finished water storage and distribution. This disinfection strategy minimizes free chlorine contact and the resultant production of DBPs. Concentrations of TTHM and HAA5 have been low, and compliance with the Stage 2 DBPR has been safely maintained.

The West Plant uses free chlorine for oxidation of manganese and initial disinfection. Much longer free chlorine CT is provided through the detention basins of the West Plant than is provided in the East Plant. Additional disinfection is provided by chloramines through the filter clearwells, finished water storage, and distribution. The longer free chlorine contact through the detention basins does not form greater concentrations of TTHM and HAA5 in the West Plant than the East Plant due to the lower TOC concentrations in the West Plant.

Distribution TTHM and HAA5 concentrations have averaged 24.9 and 9.6 µg/L over the past 3 years, respectively. LWS is on reduced monitoring for DBPs and samples three sites each for TTHM and HAA5. Maximum TTHM and HAA5 concentrations of 40.7 and 22.9 µg/L have been detected, respectively. The MCLs for TTHM and HAA5 are 80 and 60 µg/L, respectively. Compliance with the MCLs is based on a locational running annual average (LRAA), recently implemented in the Stage 2 DBPR. LWS has sampled and monitored 12 sites for TTHM/HAA5 compliance monitoring throughout the distribution system. Monitoring at many of these sites has just recently begun in 2012-2013 with the start of the Stage 2 DBPR. The highest LRAA at any of the 12 monitoring sites over the past 3 years has been 26.0 and 18.0 µg/L for TTHM and HAA5, respectively. Therefore, the LRAA for both TTHM and HAA5 has been less than half the MCL over the past 3 years and well within compliance of the recent Stage 2 DBPR. Even the maximum TTHM and HAA5 concentrations are well below the LRAA MCLs.

3.1.5 Bromate

Bromate is a DBP currently regulated under the Stage 1 DBPR. Bromate is formed by reaction of ozone with bromide ion. The East Plant uses ozone for oxidation of manganese and initial disinfection, and bromate is produced as a byproduct of the treatment process. Concentrations of bromate have been low, and compliance with the Stage 1 DBPR has been safely maintained.

Bromate concentrations have averaged 0.9 μg/L over the past 3 years. A maximum bromate concentration of 4.5 μg/L has been monitored. The MCL for bromate is 10 μg/L. Therefore, bromate concentrations have been less than half the MCL over the past 3 years and well within compliance of the Stage 2 DBPR. LWS is currently on reduced quarterly monitoring.

3.1.6 Arsenic

Arsenic is a naturally occurring inorganic contaminant in the raw groundwater. Arsenic concentrations have averaged 8.0 and 7.0 μg/L over the past 3 years in the East Plant and West Plant, respectively. Maximum arsenic concentrations of 8.9 and 7.5 μg/L have been monitored in the East Plant and West Plant, respectively. The MCL for arsenic is 10 μg/L. Therefore, arsenic concentrations have been very consistent, and LWS has been in compliance with the MCL.

3.1.7 Atrazine

Atrazine is an agricultural herbicide that has been declining in use and occurrence in the raw groundwater through the years. Ozonation has been used in the East Plant for oxidation and destruction of atrazine to maintain compliance with higher average and maximum raw atrazine concentrations over the years.

Raw water atrazine concentrations have averaged 0.70 and 0.28 μg/L over the past 3 years in the East Plant and West Plant, respectively. Maximum raw water atrazine concentrations of 4.4 and 0.52 μg/L have been detected in the East Plant and West Plant, respectively. The atrazine levels in the East Plant are much higher and more variable than the West Plant due to the influence of the HCWs as groundwater under the direct influence of surface water. With a maximum atrazine concentration of 4.4 μg/L in the East Plant raw water, treatment with ozone to reduce the atrazine concentration below the finished water MCL is still necessary.

LWS manages the selection of specific wells to minimize atrazine levels coming into the East Plant. Finished water atrazine concentrations have averaged 0.09 and 0.10 μg/L over the past 3 years in the East Plant and West Plant, respectively. Maximum finished water atrazine concentrations of 0.27 and 0.22 μg/L have been monitored in the East Plant and West Plant, respectively. The MCL for atrazine is 3 μg/L. Therefore, finished water atrazine concentrations have been well below the MCL, and LWS has been in compliance with the regulations.

3.2 Current Regulatory Framework

The U.S. Environmental Protection Agency (EPA) has finalized 19 drinking water regulations since 1975, nine prior to the 1996 Safe Drinking Water Act (SDWA) Amendments and ten after 1996. Several of the early rules have been revised, some of them more than once. The later rules are generally more complex in nature with multiple requirements and deadlines. Table 3-4 lists the regulations and the basic requirements of each rule. LWS is consistently in compliance with the current regulations.

Table 3-4 Current Drinking Water Regulations

SDWA Regulation	Compliance Date	General Requirements
National Interim Primary Drinking Water Regulations	1976	Set first MCLs for inorganic and organic chemicals, turbidity, total coliform and radioactive constituents. Mercury, nitrate, and selenium MCLs still stand, others have been revised.
Lead and Copper Rule (LCR) and Revisions to LCR	1992 & 2000	Ensure pH control and other corrosion control strategies are appropriate to meet action levels.
Phase I, Phase II and Phase V Synthetic and Volatile Organic Chemicals	1987, 1991, 1992	Multiple requirements for monitoring and removal to MCL levels for organic chemicals
Surface Water Treatment Rule (SWTR)	1989	Disinfection requirements continue in force although turbidity superseded by Interim Enhanced Surface Water Treatment Rule (IESWTR); 4-log removal of viruses, 3-log removal of *Giardia*
Total Coliform Rule (TCR)	1990	Ensure disinfection strategy and pH control to maintain distribution system water quality; weekly monitoring in distribution system
Interim Enhanced Surface Water Treatment Rule (IESWTR)	Jan 2002	Combined filter effluent turbidity of 0.3 NTU 95 percent of time, not to exceed 1 NTU; continuous monitoring of filters
Stage 1 Disinfectant/ Disinfection Byproduct Rule (DBPR)	Jan 2002	Meet TTHM/HAA5 $\leq$ 80/60 µg/L; disinfectant MRDLs; TOC removal; monitoring plan
Filter Backwash Rule	Dec 2003	Notify WDEQ of recycle practices; return all recycle flow to head of plant
Radionuclides	Dec 2003	Meet MCLs for radioactive contaminants
Arsenic	Jan 2006	Meet MCL for arsenic
Ground Water Rule	Dec 2009	Maintain 4-log inactivation of viruses to avoid source water monitoring that is triggered by total coliform positive sample in distribution system

SDWA Regulation	Compliance Date	General Requirements
Stage 2 Disinfectant/ Disinfection Byproduct Rule (DBPR)	October 2012	Initial Distribution System Evaluation required to determine monitoring sites; Meet TTHM/HAA5 $\leq$ 80/60 µg/L based on locational running annual averages [1]
Long-term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR)	October 2012 - Treatment Technique October 2015 – 2nd round of sampling[2]	Monitor for *Cryptosporidium* to determine treatment requirement and provide additional treatment if required; disinfection profiling
Revised Total Coliform Rule (RTCR)	April 2016	Continue monitoring for total coliform at same level as TCR; if exceed TC-positive trigger, complete appropriate assessment to find and fix the total coliform problem

Notes:
1. *LWS has been granted approval by Nebraska Department of Health and Human Services (NDHHS) for reduced monitoring for disinfection byproducts.*
2. *LWS is scheduled to begin sampling in April 2015.*

3.2.1 Revised Total Coliform Rule

The Revised Total Coliform Rule (RTCR) was finalized in April 2013. The RTCR shifts the regulatory focus from public notification for total coliform positive occurrences to a “find and fix” framework. Two tiers of assessment requirements in the rule are set to require either the utility or a third party to complete an evaluation to determine the cause of the total coliform positive results and to make changes to eliminate them in the future.

3.2.2 New “Lead Free” Definition

Public Law 111-380 revised the definition of “lead-free” from lead less than 8.0 percent to less than 0.25 percent for pipes, fixtures, and appurtenances (meters, pipe fittings, etc.). Saddles, meters, and parts all had to meet the new definition by January 4, 2014. This required water systems to manage their existing inventory to meet the 2014 deadline.

3.2.3 Electronic Delivery of Consumer Confidence Reports

EPA “interpretive memo” on methods of delivering consumer confidence reports (CCR) was issued in January 2013. The consumer confidence reports must be mailed or directly delivered to customers. The interpretive memo provides an overview of electronic delivery methods, and clarifies that the following methods of delivery are acceptable:

- URL sent on postcard
- URL sent in customer bill
- URL has to go directly to the CCR

3.3 Future Regulations

3.3.1 Pending Regulations

Likely regulatory actions occurring in the 2014-2015 timeframe will come from the preliminary Third Regulatory Determination, the proposed Long-Term Lead and Copper Rule, the proposed carcinogenic Volatile Organic Compounds Rule, or the proposed Perchlorate Rule. Actions further out in time will arise from the third 6-year review process or from separate actions directed by legislation.

3.3.2 Third Regulatory Determination

Over the next 5 to 10 years, the regulatory changes coming from EPA depend somewhat on the decisions made based on the Third Regulatory Determination process. EPA is required to make decisions (regulatory determinations) on at least five contaminants every 5 years. The decisions can be negative (no regulation needed) or positive determinations. A positive determination means that EPA is going to move ahead to develop a drinking water rule, and if positive determinations are made for a number of contaminants, the schedule could slip because it creates more work for the EPA staff. The preliminary Third Regulatory Determination was due in late 2013 and will be finalized in 2014 – 2015. The Federal budget may impact the determinations.

Potential positive determinations from the Third Regulatory Determination include nitrosamines (Nitrosodimethylamine [NDMA] is the most prevalent nitrosamine), chlorate, and strontium. Positive determinations require that EPA propose a drinking water regulation within 24 months of the final regulatory determination and a final rule 18 months later. This means that regulations for nitrosamines, chlorate, or strontium would not be finalized until 2018 to 2020.

At this point in time, the regulatory MCL target for nitrosamines, or NDMA as a surrogate for the group of nitrosamines, is not defined. In fact, the justification for regulating NDMA in drinking water will be a difficult issue for EPA. The Safe Drinking Water Act requires EPA to demonstrate a meaningful reduction in public health risk in order to regulate a contaminant. While NDMA is a known carcinogen and it does occur at low levels in drinking water (also requirements for regulation), the contribution of drinking water to the total exposure of NDMA is very low (0.02 percent of the total exposure from external sources and endogenous generation of NDMA), so the reduction in risk by regulating NDMA in drinking water will be minimal. The quantification of NDMA by high resolution GC/MS is approximately 2 ng/L so the regulatory level will not be below that level. International guidelines (World Heath Organization [WHO], Australia and Canada) range from 40 ng/L to 100 ng/L for NDMA. A maximum concentration of 2.8 ng/L has been observed in 2008-2009 sampling by LWS.

3.3.3 Proposed Perchlorate Rule

The perchlorate rule has a checkered past with EPA because the original determination for perchlorate was negative (no regulation needed). Then due to various pressures, EPA reversed that decision and made a positive determination to regulate. Along with that pressure came a statutory deadline to propose a perchlorate rule by February 2013, which was missed by EPA. Multiple scientific and technical issues have been raised by a special committee of the Science Advisory Board (SAB), which was consulted by EPA on perchlorate issues. The SAB recommended setting an MCLG (maximum contaminant level goal) for perchlorate but with additional analyses required. The SAB final report is expected to be delivered to EPA in 2013. Setting an MCLG is complex with respect to iodide deficiency, dose/response needed to change iodide levels, the use of EPA's pharmacokinetic/pharmacodynamic model, and the incorporation of "life stages" (infants) in the analysis.

Perchlorate health advisory levels have been set by a number of states, ranging from 1 to 18 μg/L. Potential regulatory levels for a national rule range from 2 to 10 μg/L. The rule proposal may be delayed until 2015. LWS sampled and analyzed perchlorate as part of UCMR sampling; no perchlorate was detected in any sample.

3.3.4 Proposed Long-Term Lead and Copper Rule

EPA has been working on the content of the Long-Term Lead and Copper Rule for some time, but the regulatory proposal is not expected until sometime in 2014. The Revisions will likely include some requirements related to partial lead service line replacement (PLSLR). Research results are not clear whether PLSLR increases or decreases lead exposure, so EPA may hold stakeholder outreach to address the complex scientific and technical issues. Also included in the Revisions will be requirements for optimization of corrosion control and water quality parameters. Sample site selection criteria may be altered to include schools and day care centers and may shift the focus to homes with lead service lines with less attention to lead solder. Sampling protocols may change to represent water that has "overnighted" in lead service lines as opposed to first flush samples. LWS is in compliance with current Lead and Copper Rule and is on tri-annual sampling schedule.

3.3.5 Proposed Carcinogenic Volatile Organic Compounds Rule

EPA is currently collecting more occurrence and treatment data to determine which carcinogenic volatile organic compounds (cVOCs) to include in the regulation. The intent is to construct a rule that regulates a group of contaminants, but at this point which contaminants will be include is still in question. They plan to build on the existing risk assessments for TCE and PCE, so these two compounds are very likely to be included in the regulated group. Potential co-occurrence and common treatment will be considered by EPA in selecting the cVOCs. TCP (1,2,3-trichloropropane) is likely to be included because it is highly carcinogenic. A rule proposal is expected in 2014. None of the proposed compounds have been detected in LWS samples.

3.3.6 Potential Future Regulations – Regulatory Actions Beyond 2015

The SDWA requires EPA to review all drinking water regulations every 6 years for possible revision. New information pertaining to health effects, analytical methods, occurrence, or treatment data can lead EPA to include a rule on the list for revision. Preliminary notice of the rules that EPA expects to revise is due in 2015, with the finalized list of rules for revision in 2016-2017. Actual revisions would then be proposed and finalized in the 2020 – 2025 timeframe. Expectations are that the following rules will be included for revision: the Stage 2 Disinfection Byproduct Rule, Interim Enhanced Surface Water Treatment Rule, and the Long-Term 2 Enhanced Surface Water Treatment Rule.

In addition, chromium is likely to be included as part of the 6-year review list because hexavalent chromium (Cr-6) is of concern. EPA is developing a Cr-6 toxicological review now in preparation for the rule revision. Both Cr-6 and total chromium are part of the Third Unregulated Contaminant Monitoring (UCMR3) program which is currently underway. Systems are required to sample finished water at entry points and at the maximum residence time locations in the distribution system at a detection level of 20 ng/L for Cr-6 (not to be confused with an MCL). Widespread occurrence is expected and a regulation is likely that includes both total chromium (currently regulated) and hexavalent chromium. A target MCL has not been determined for Cr-6. A maximum concentration of 130 ng/L has been monitored in 2011 sampling by LWS.

For an idea of what compounds might be regulated further out in time, one source is the list of contaminants on the UCMR3 monitoring program. That list includes a few metals and some VOCs, several perfluorocarbons, 1,4-dioxane, two viruses and seven hormones, along with total chromium and hexavalent chromium. The evaluation of occurrence of contaminants is one of the criteria for EPA to regulate – if a contaminant does not occur in drinking water in significant amounts, it will not be regulated. LWS should track the results of the UCMR3 monitoring to understand which contaminants may be candidates for regulation in the future. The UCMR3 monitoring occurs from 2013-2015, so by the end of 2016, data should become available and will be discussed and presented at conferences and in regulatory discussions. LWS will be sampling in March, June, September, and December 2015.

4.0 Water Treatment Plant Improvement Evaluation

4.1 Future Capacity Needs

Chapter 2 -Water Capacity Requirements presented system demand projections for the design years 2025, 2040, and 2060, as well as decennial years in the planning horizon. The demand projections are based on the population forecasts, the per-capita demand, and the peaking factors for the maximum day and the maximum hour previously presented. Figure 4-1 graphically depicts the demand projections.

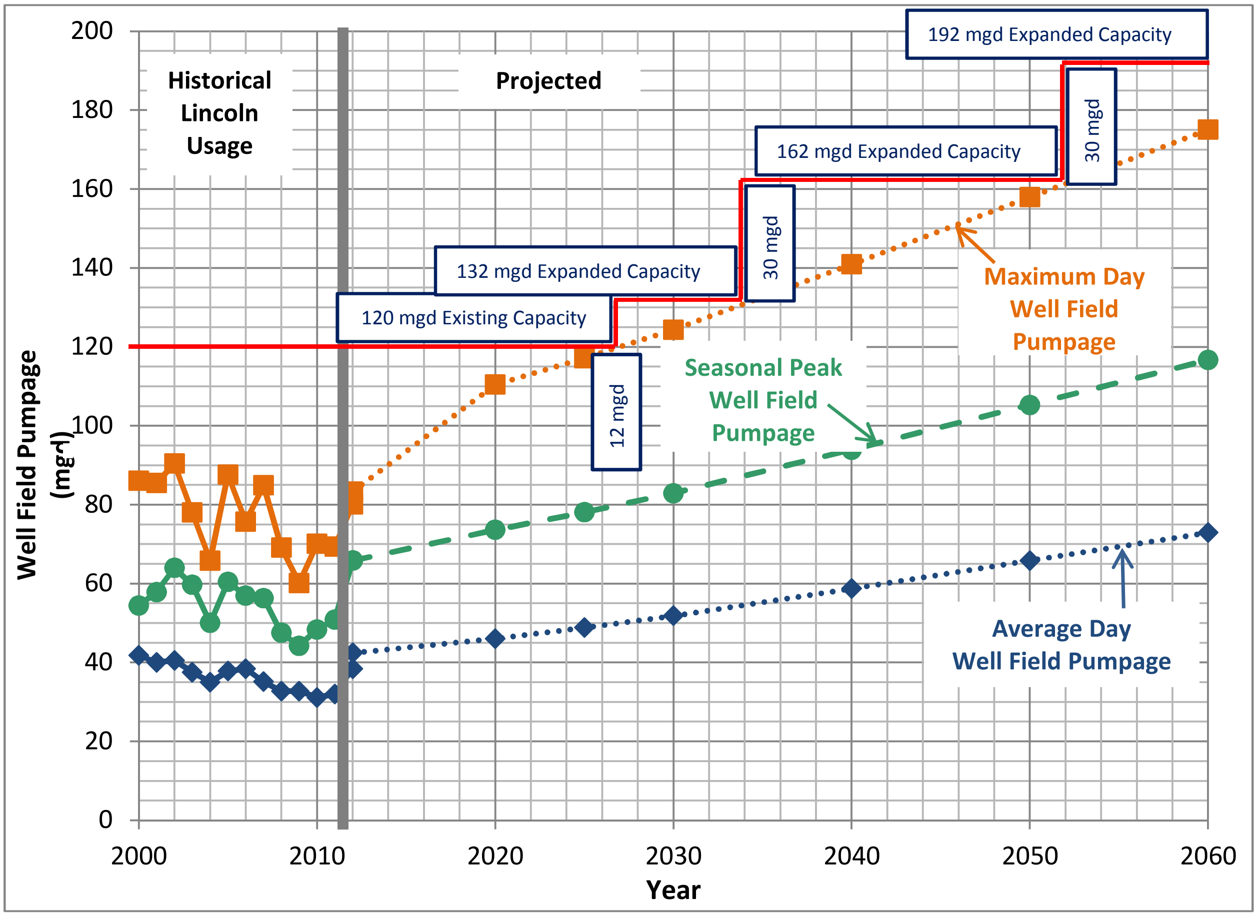

Figure 4-1 Future Demand Projections

4.2 Plant Expansion Needs

As indicated in Figure 4-1, the 120 MGD existing treatment capacity of the combined East and West Plants is projected to be exceeded in Year 2027. Planning for design and construction of capacity improvements should be undertaken by Year 2022. A 12 MGD capacity expansion of the West Plant is discussed below. The 12 MGD expansion to a total capacity of 132 MGD would meet projected demands for 7 more years until Year 2034. Planning for design and construction of the next capacity expansion should be undertaken by Year 2029. A 30 MGD expansion of the East Plant is discussed below. The 30 MGD expansion to a total capacity of 162 MGD would meet projected demands for 18 more years until Year 2052. Planning for design and construction of the next capacity expansion should be undertaken by Year 2047.

Another 30 MGD expansion of the East Plant is discussed below. The 30 MGD expansion to a total capacity of 192 MGD would meet projected demands for several years beyond the Year 2060 planning window of this Master Plan. The need for the second East Plant expansion is dependent on the timing of implementation of the Missouri River Project. See Table 4-1 below for a summary of the recommended plant expansions.

Table 4-1 Recommended Plant Expansions

Expansion Year	Expansion Description	Expansion Capacity	Total Capacity
2027	West Plant Expansion	12 MGD	132 MGD
2034	First East Plant Expansion	30 MGD	162 MGD
2052	Second East Plant Expansion	30 MGD	192 MGD

4.2.1 12-MGD West Plant Expansion

Planning for design and construction of a 12 MGD capacity expansion of the West Plant should be undertaken by Year 2022 for the improvements to be in service by Year 2027 when the existing 120 MGD capacity is projected to be exceeded. A total capacity of 132 MGD would be provided with these improvements, which would meet projected demands for 7 more years until Year 2034.

4.2.2 First 30-MGD East Plant Expansion

Planning for design and construction of an initial 30 MGD capacity expansion of the East Plant should be undertaken by Year 2029 for the improvements to be in service by Year 2034 when the 132 MGD capacity is projected to be exceeded. A total capacity of 162 MGD would be provided with these improvements, which would meet projected demands for 18 more years until Year 2052.

4.2.3 Second 30-MGD East Plant Expansion

Planning for design and construction of a second 30 MGD capacity expansion of the East Plant should be undertaken by Year 2047 for the improvements to be in service by Year 2052 when the 162 MGD capacity is projected to be exceeded. A total capacity of 192 MGD would be provided with these improvements, which would meet projected demands for several years beyond the Year 2060 planning window of this Master Plan.

4.2.4 Missouri River Project and Treatment

Chapter 3 -Water Supply of this Master Plan evaluates long-term water supply options. Once the third and fourth HCWs are constructed, the projected supply deficit in Year 2060 is 50 MGD of instantaneous and short-term pumping capacity to meet the maximum day demand, and 35 MGD of summer yield capacity to meet the seasonal demand. Only two supply sources, the Missouri River and Platte River alluvial aquifers, were identified as viable options for the development of large scale, reliable water supplies. An alternatives analysis identified the Missouri River as the preferred water supply source for the additional long-term supply and the addition of fifth and sixth HCWs for the mid-term supply. The conclusions also indicated that the Missouri River Project would be able to support a 60 MGD demand even during significant drought conditions. Additionally, it was noted that having separate sources of supply from the different rivers would result in source diversification of the raw water supply, which would provide operational flexibility to the City. The Missouri River Project considered in this Master Plan included a new water treatment facility located near the well field developed along the Missouri River.

Further Platte River alluvial aquifer expansion, as discussed in *Chapter 3,* would result in 155 MGD maximum instantaneous raw water capacity during low flow conditions. The first 30 MGD East Plant expansion would result in 162 MGD treatment plant capacity. If the Missouri River alluvial aquifer source and treatment option is implemented after this in 2045, then the second 30 MGD East Plant expansion would not be constructed.

4.3 Regulatory Improvements

Both East and West Plants are in compliance with all existing drinking water regulations. Rulemaking in the foreseeable future does not appear to adversely impact the plants. However, if more stringent National Pollutant Discharge Elimination System (NPDES) discharge permit limits for filter backwash water and filter-to-waste water would be imposed, then improvements may be necessary. These and other potential improvements are discussed in the following sections.

4.3.1 NPDES Discharge Compliance Improvements

Filter backwash waste and filter-to-waste flow streams are currently directly discharged to a small stream tributary. The current NPDES discharge permit has limits for pH and residual chlorine, for which compliance will be provided by improvements currently in process. If NPDES discharge permit limits for flow rate, suspended solids, manganese, or iron are imposed at limits below the current discharge levels, then further improvements may be necessary. Flow attenuation to hold the high backwash and filter-to-waste flow rates and volumes, and discharge more continuously over longer periods at lower flow rates may be required. Treatment of the flow streams to remove suspended solids of manganese and iron precipitates may be required. It is not anticipated that pH adjustment of the discharge would be required, as permit limits are typically within the range of pH 6-9 and the discharge is in the range of pH 7-8.

Flow attenuation and initial settling is typically provided by basins of adequate volume to hold the required number of filter backwash and filter-to-waste cycles with decant from the basins at lower continuous flow rate recycled back to the head of the treatment plant or directly discharged to the receiving stream. Removal of suspended solids can be efficiently provided by plate settlers, which allow for a high degree of solids removal at a high upflow rate and small footprint area compared to traditional clarifiers.

Storage and ultimate disposal of the solids separated from the attenuated flow stream would be required. Temporary storage of the solids in residual storage lagoons is typically provided if adequate land area is available. The manganese and iron precipitates do not dewater well using mechanical dewatering processes such as presses, so mechanical dewatering is not feasible. The manganese and iron precipitates are not beneficial for land application like lime softening solids, so ultimate disposal by land application is more challenging. More land area for land application would be required to keep application rates low to minimize manganese and iron loading to the soil.

4.3.2 UV Disinfection Improvements

LWS previously conducted 3 years of monitoring that resulted in no detection of *Cryptosporidium* in the raw water. This placed LWS in Bin 1 of the LT2ESWTR, requiring no further action. Another 3 years of monitoring for *Cryptosporidium* will begin in 2015. The results of this additional monitoring will determine whether or not further disinfection or treatment action is necessary. Since *Cryptosporidium* was not detected in the previous sampling, even in the HCW groundwater under the direct influence of surface water, it is not anticipated that it will be detected in this next round of sampling. Then no further action or improvements would be necessary.

However, if *Cryptosporidium* is detected at a level placing LWS in Bin 2 or greater, then additional improvements will be required. The Microbial Toolbox of the LT2ESWTR provides many options for additional log credit from 0.5-log credit to >2.5-log credit, depending on the additional log treatment necessary by the Bin requirements.

Ultraviolet (UV) disinfection of the filtered water is one option that could provide greater than 2.5-log credit if necessary. UV is not anticipated to be required or recommended in the foreseeable future.

4.3.3 Membrane Improvements

Membrane filtration is another option that could provide greater than 2.5-log credit if necessary to meet Bin 2 or greater requirements of the LT2ESWTR. Membrane filtration could also replace granular filtration in either plant. However, membranes would be much more costly than UV disinfection and more complicated to operate than the current granular filtration. Therefore, membrane filtration is not anticipated to be required or recommended in the foreseeable future.

4.3.4 Granular Activated Carbon Improvements

Granular activated carbon (GAC) is effective in the removal of organics such as NOM, taste and odor compounds, and synthetic or volatile organic contaminants by adsorption. GAC could be implemented either in a new post-filter facility with GAC contactors or in replacement of anthracite with GAC in the existing filters as filter/adsorbers. Retrofit of the existing filters to filter/adsorbers with GAC would likely involve other peripheral renovations to facilitate the change. There is no regulatory driver to further reduce NOM for DBP compliance or otherwise, or reduce any specific synthetic or volatile organic contaminants. There is also no need for GAC for removal of objectionable taste or odor compounds. Treatment with GAC may be required if the contaminants from the former Ft. Mead munitions productions facility break through the groundwater clean-up and containment systems. GAC is not anticipated to be required or recommended in the foreseeable future.

4.3.5 Ozone for West Plant Improvements

The East Plant uses ozone for manganese oxidation and initial disinfection, while the West Plant uses chlorine for both manganese oxidation and initial disinfection. Both plants perform effectively for manganese removal, turbidity removal, disinfection, and DBP control while meeting all current and anticipated drinking water regulations. There is no driver, either regulatory or cost-wise, to add ozone to the West Plant process. Capacity expansions of the East Plant should continue to use ozone, up to the ultimate 150 MGD capacity. The West Plant should continue to use chlorine at a slightly expanded capacity of 72 MGD within its current footprint.

5.0 Recommended Improvements

5.1 Capacity Expansion Improvements

Three phases of capacity expansion improvements are recommended throughout the planning period based on the water demand projects developed in this Master Plan. The three recommended projects with opinions of probable costs are as follows:

5.1.1 12-MGD West Plant Expansion

Planning for design and construction of a 12 MGD capacity expansion of the West Plant should be undertaken by Year 2022 for the improvements to be in service by Year 2027 when the existing 120 MGD capacity is projected to be exceeded. A total capacity of 132 MGD would be provided with these improvements, which would meet projected demands for 7 more years until Year 2034.

Elements of the recommended improvements include the following:

- Filter pilot testing of alternative dual-media gradations and configurations to determine the final recommended dual-media system, filtration rate, backwashing procedure, control scheme, and any other related improvements
- Filter dual-media conversion of all West Plant filters
- SDS TTHM and HAA5 testing to determine what amount of HCW water could be used in the West Plant while safely maintaining compliance with the Stage 2 DBPR
- New sodium hypochlorite generation system to replace gaseous chlorine storage and feed system
- New aqueous ammonia system to replace anhydrous ammonia storage and feed system

An opinion of probable cost for these improvements is presented in Table 5-1 below.

Table 5-1 Opinion of Probable Cost – 12 MGD West Plant Expansion

Year[1]	Description	Current Cost Basis[2]	Future Cost Basis - 3% Inflation[4]	Future Cost Basis - 5% Inflation[5]
2022	Filter Pilot Testing	$50,000	$65,000	$78,000
2022	SDS TTHM & HAA5 Testing	$50,000	$65,000	$78,000
2027	Filter Dual-Media Conversion[3]	$3,750,000	$5,674,000	$7,425,000
2027	New Sodium Hypochlorite Generation System	$6,300,000	$9,532,000	$12,474,000
2027	New Aqueous Ammonia System	$270,000	$409,000	$535,000
2027	Subtotal Construction Cost	$10,420,000	$15,745,000	$20,590,000
2027	Contingency (25%)	$2,605,000	$3,936,000	$5,148,000
2027	Engineering (15%)	$1,563,000	$2,362,000	$3,089,000
2027	**Total Project Cost**	**$14,588,000**	**$22,043,000**	**$28,827,000**

Notes:
1. *Years indicated are years when new capacity should be operational.*
2. *2013 dollars.*
3. *Current cost based on $400/ft^2$ of filter area for new media, troughs, and underdrains if necessary.*
4. *Inflated to projected year dollars at 3% per year inflation rate.*
5. *Inflated to projected year dollars at 5% per year inflation rate.*

5.1.2 First 30-MGD East Plant Expansion

Planning for design and construction of an initial 30 MGD capacity expansion of the East Plant should be undertaken by Year 2029 for the improvements to be in service by Year 2034 when the 132 MGD capacity is projected to be exceeded. A total capacity of 162 MGD would be provided with these improvements, which would meet projected demands for 18 more years until Year 2052.

An opinion of probable cost for these improvements is presented in Table 5-2 below.

Table 5-2 Opinion of Probable Cost – First 30 MGD East Plant Expansion

Year[1]	Description	Current Cost Basis[2]	Future Cost Basis - 3% Inflation [4]	Future Cost Basis - 5% Inflation[5]
2034	30 MGD East Plant Expansion[3]	$18,000,000	$33,480,000	$50,148,000
2034	Subtotal Construction Cost	$18,000,000	$33,480,000	$50,148,000
2034	Contingency (25%)	$4,500,000	$8,370,000	$12,537,000
2034	Engineering (15%)	$2,700,000	$5,022,000	$7,522,000
2034	**Total Project Cost**	**$25,200,000**	**$46,872,000**	**$70,207,000**

Notes:
1. *Years indicated are when new capacity should be operational.*
2. *2013 dollars.*
3. *Current cost based on HDR Project Cost Estimating software.*
4. *Inflated to projected year dollars at 3% per year inflation rate.*
5. *Inflated to projected year dollars at 5% per year inflation rate.*

5.1.3 Second 30-MGD East Plant Expansion

Planning for design and construction of a second 30 MGD capacity expansion of the East Plant should be undertaken by Year 2047 for the improvements to be in service by Year 2052 when the 162 MGD capacity is projected to be exceeded. A total capacity of 192 MGD would be provided with these improvements, which would meet projected demands for several years beyond the Year 2060 planning window of this Master Plan. As discussed above, the timing of the Missouri River Project may impact the need for this second East Plant expansion.

An opinion of probable cost for these improvements is presented in Table 5-3 below.

Table 5-3 Opinion of Probable Cost – Second 30 MGD East Plant Expansion[1]

Year[2]	Description	Current Cost Basis[3]	Future Cost Basis - 3% Inflation [4]	Future Cost Basis - 5% Inflation[5]
2052	30 MGD East Plant Expansion[6]	$17,000,000	$53,992,000	$113,985,000
2052	Subtotal Construction Cost	$17,000,000	$53,992,000	$113,985,000
2052	Contingency (25%)	$4,250,000	$13,498,000	$28,496,000
2052	Engineering (15%)	$2,550,000	$8,099,000	$17,098,000
2052	**Total Project Cost**	**$23,800,000**	**$75,589,000**	**$159,579,000**

Notes:
1. *The need for this expansion may be impacted by the timing of the Missouri River Project*
2. *Years indicated are when new capacity should be operational.*
3. *2013 dollars.*
4. *Inflated to projected year dollars at 3% per year inflation rate.*
5. *Inflated to projected year dollars at 5% per year inflation rate.*
6. *Current cost based on HDR Project Cost Estimating software.*

5.2 Regulatory Improvements

Only NPDES discharge permit compliance improvements are anticipated to possibly be required throughout the planning period. The other potential regulatory improvements discussed previously are not anticipated at this time.

5.3 Summary of All Improvements

The various recommended improvements for capacity expansion, regulatory compliance, and energy efficiency are summarized in Table 5-4 as follows.

Table 5-4 Opinion of Probable Cost – Summary of All Improvements

Year[1]	Description	Current Cost Basis[2]	Future Cost Basis - 3% Inflation [3]	Future Cost Basis - 5% Inflation[4]
2027	12 MGD West Plant Expansion[5]	$14,588,000	$22,043,000	$28,827,000
2034	First 30 MGD East Plant Expansion	$25,200,000	$46,872,000	$70,207,000
2052	Second 30 MGD East Plant Expansion[6]	$23,800,000	$75,589,000	$159,579,000
	Total of All Projects	**$63,588,000**	**$144,504,000**	**$258,613,000**

Notes:
1. *Years indicated are when new capacity should be operational.*
2. *2013 dollars.*
3. *Inflated to projected year dollars at 3% per year inflation rate.*
4. *Inflated to projected year dollars at 5% per year inflation rate.*
5. *Testing for rerating/expansion of the west plant is recommended in 2022. See Table 5-1 for more detail.*
6. *The timing of the Missouri River Project may impact the need for the second East Plant expansion.*

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Lincoln Water System Facilities Master Plan

Chapter 5 - Transmission and Distribution System

May 2014

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Abbreviations and Acronyms

2007 Master Plan	2007 Facilities Master Plan
AACE International	Association for the Advancement of Cost Engineering International
ADD	average day demand
AMD	average month demand
CFD	computational fluid dynamics
CIP	Capital Improvements Program
City	City of Lincoln
FY	fiscal year
gpcd	gallons per capita per day
GIS	geographic information system
gpm	gallons per minute
ID	identification
LES	Lincoln Electric System
LPlan 2040	Lincoln/Lancaster County 2040 Comprehensive Plan
LWS	Lincoln Water System
Master Plan	2013 Facilities Master Plan
MDD	maximum day demand
MG	million gallons
MGD	million gallons per day
MHD	maximum hour demand
MMD	minimum month demand
OPPD	Omaha Public Power District
psi	pounds per square inch
PRV	pressure reducing valve
SCADA	Supervisory Control and Data Acquisition
SP	Seasonal Peak
TAZ	transportation analysis zone

TOU	time of use
USGS	United States Geological Survey
VFD	variable frequency drive
WTP	Platte River Water Treatment Facility

1.0 Introduction

Chapter 5 - Transmission and Distribution Systems presents a plan for distribution system improvements that will maintain and expand service to customers as the City of Lincoln (City) grows over the next 50 years. The improvements are based on delivery of the water usage projections determined in *Chapter 2* of the 2013 Facilities Master Plan (Master Plan) while meeting evaluation criteria as documented later in this chapter. The *Chapter 5* planning effort includes:

- Summarizing the existing transmission and distribution systems infrastructure and operations
- Updating, validating, and upgrading the existing distribution system computer model
- Analyzing the distribution system capacity using the updated computer model
- Evaluating system water age as an indicator of general water quality
- Completing analyses of specific infrastructure for operational and efficiency improvements
- Summarizing recommended short-term and long-term transmission and distribution systems improvements

2.0 Basis of Planning

2.1 Planning Period

The planning period for this master planning effort is from the year 2014 through the year 2060. This planning period is used for evaluating system improvements or service expansions. Year 2012 serves as the base year for this analysis. The distribution system analysis is conducted for three specific planning intervals:

- 2025 (short-term, 2014-2025)
- 2040 (mid-term, 2026-2040)
- 2060 (long-term, 2041-2060)

These planning periods were selected in coordination with the Lincoln/Lancaster County 2040 Comprehensive Plan (LPlan 2040).

2.1.1 2025 (Short-term, 2014-2025)

The short-term analyses provide recommendations for both improvements to address existing system deficiencies and expanding facilities to serve short-term new development areas. For this time frame, recommended improvements are prioritized, and construction phasing and timelines are developed. Recommended improvements are summarized in a Capital Improvement Program (CIP) along with estimated capital costs.

2.1.2 2040 (Mid-term, 2026-2040)

The mid-term analyses provide an interim benchmark between short-term facility improvements and long-term goals. These analyses provide a basis for the timing of phased improvements and provide a measure of how soon major improvements may be required after the short-term period. Recommended improvements are prioritized, and capital improvement cost estimates are provided for general planning purposes.

2.1.3 2060 (Long-term, 2041-2060)

The long-term analyses are primarily provided as a basis for evaluating how long-term growth may impact Lincoln Water System (LWS) facilities. Population projections and future development cannot be accurately quantified for this 50-year horizon, but the projections will help identify potential shortfalls in the LWS. The long-term analyses provide a basis for evaluating long-term requirements for raw water supply, treatment facility, and water transmission. The long-term plan provides a foundation for phasing of improvements and helps avoid installing short- and mid-term improvements that may not account for long-term needs. Detailed CIPs, including cost estimates, are not developed for the long-term improvements.

2.2 Study Area

The study area for the Master Plan includes the area encompassed by the City limits, both existing and future. LPlan 2040 delineates the current City limits along with the anticipated future service limits through the year 2040. In addition, two additional tiers of growth areas for beyond 2040 were delineated. As presented in Figure 2-1, these limits and growth areas include:

- **Existing City limits.** City limits as of 2011.
- **Future Service Limit through the year 2040 as defined by the Tier I growth area.** This growth area was further divided into development priority areas:
    - **Tier I – Priority A (current development):** The top priority of development is composed of undeveloped land within the existing City limits, as well as areas that are not yet annexed but that have approved preliminary plans. The Tier I – Priority A area includes approximately 22.5 square miles.
    - **Tier I – Priority B (2025):** The next priority of development is generally contiguous to existing development and should include installation of the required water system infrastructure by the year 2025. The Tier I – Priority B area encompasses approximately 17.7 square miles.
    - **Tier I – Priority C (2040):** The last priority of the Tier I growth area includes those areas that currently lack almost all infrastructure required to support urbanization. These areas are anticipated to develop towards the end of the Tier I growth period, beyond 2025, with the required water system infrastructure

being installed by the year 2040. The Tier I – Priority C area includes approximately 16.5 square miles.

- **Tier II, 50-year Long-term Potential Service Area**. Tier II is the next area of development and is anticipated to occur beyond the 30-year planning horizon of LPlan 2040 to 50 years, and possibly further. This area includes approximately 34 square miles. For the purpose of this Master Plan, this area will be considered for long-term utility planning for supply development, water treatment needs, and major infrastructure improvements through the year 2060.
- **Beyond 50-year Service Area.** LPlan 2040 defines a Tier III growth area that is beyond the planning horizon and is not considered in the Master Plan.

2.3 Service Area

Average treated water demand is typically estimated based on the total population (residential) and industrial and commercial (non-residential) served and the size and type of those areas served. Within the Study Area, the Water Service Area establishes the areas currently being served or that could be served in the short-term, mid-term, and long-term by the treated water distribution system. The existing LWS serves approximately 58,800 acres (91.9 square miles) of area in and around the City (Existing City Limits and Growth Tier I – Priority A). The proposed 2060 Water Service Area would serve approximately 102,200 acres (159.7 square miles) of area in and around the City (Existing City Limits and Growth Tiers I and II).

Figure 2-2 shows the existing and proposed Water Service Areas.

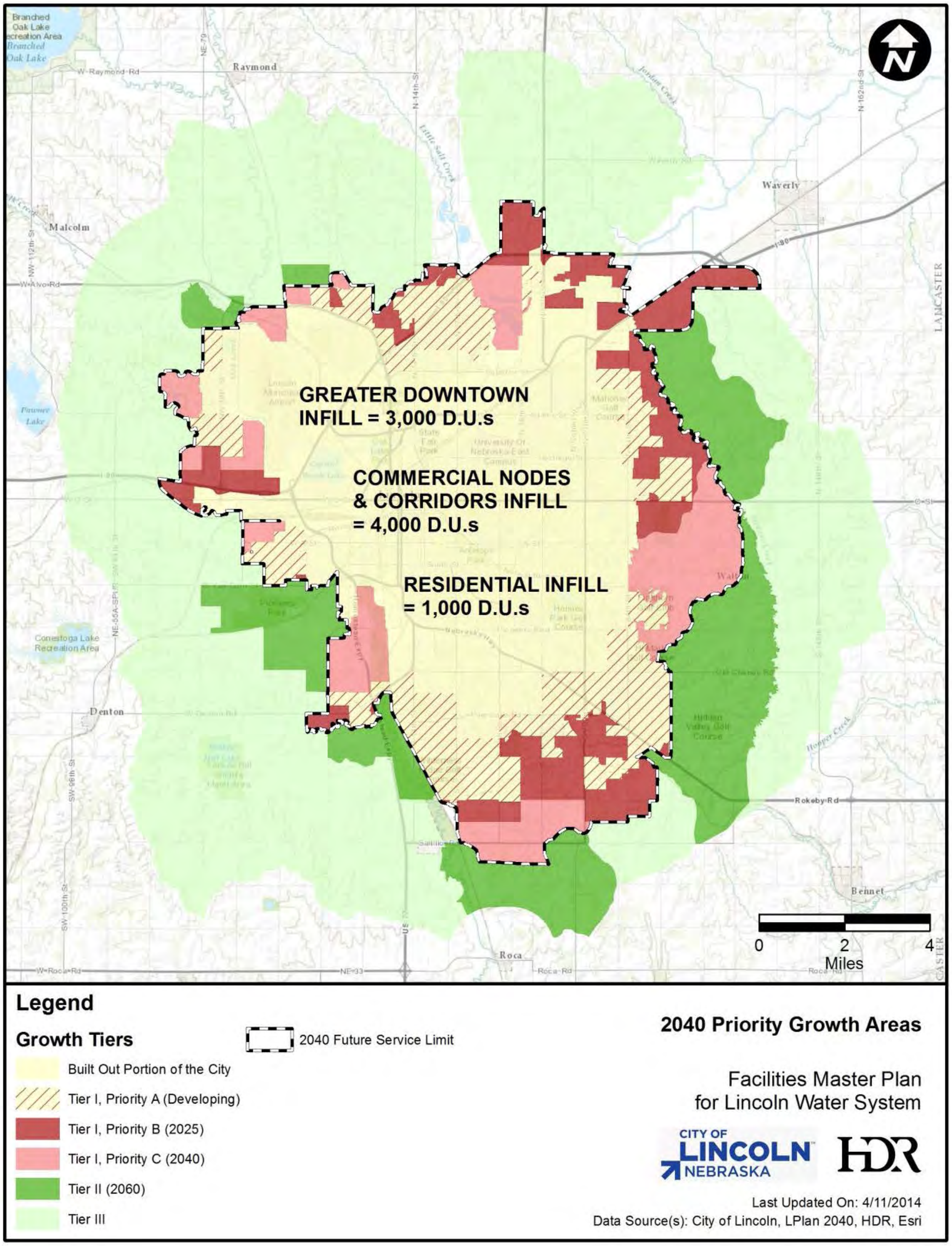

Figure 2-1 2040 Priority Growth Areas

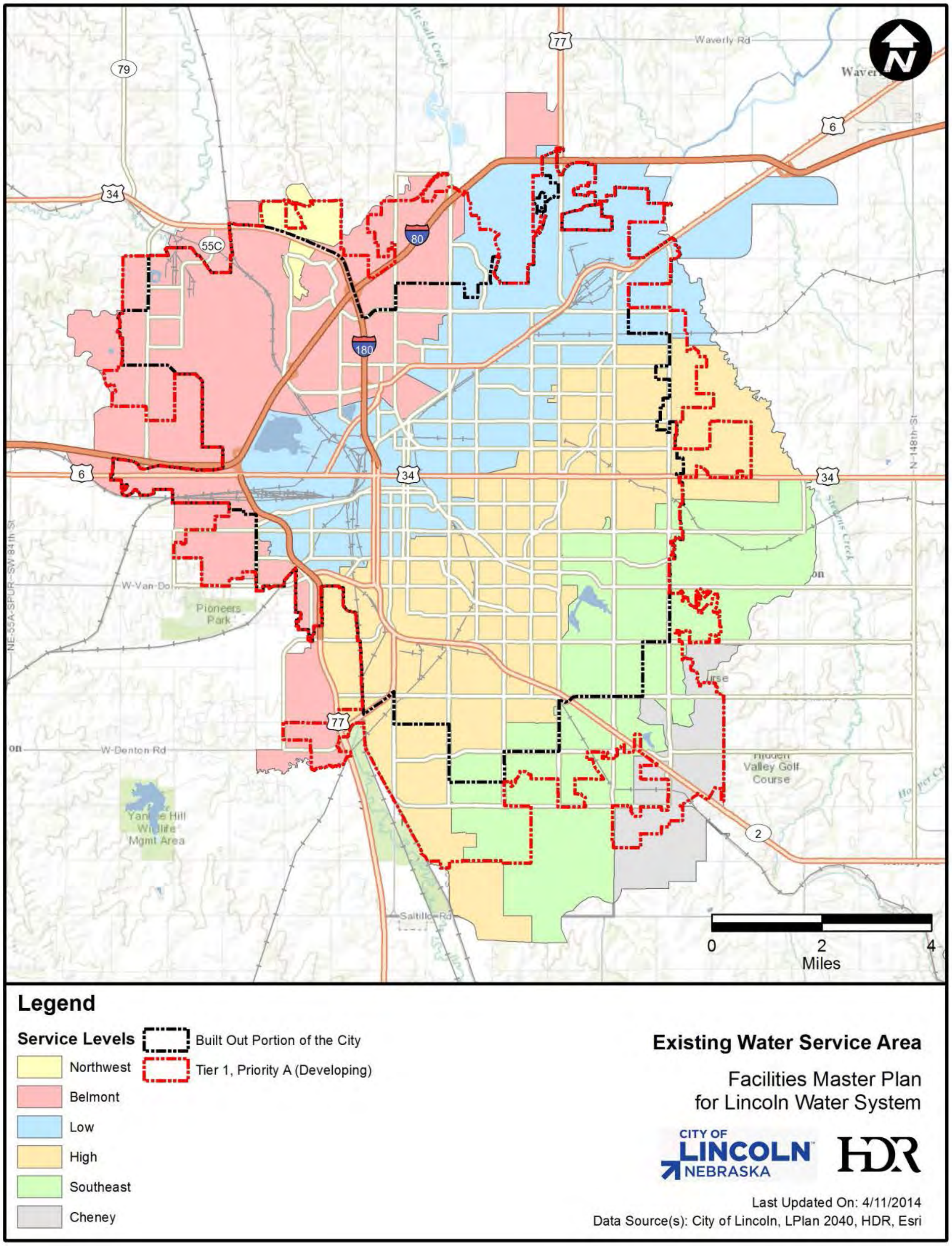

Figure 2-2 Existing Water Service Area

3.0 Existing System Facilities

The City's potable water system facilities are categorized into two major systems – transmission and distribution. The transmission system consists of transmission mains, reservoirs, and pumping stations that deliver water from the Platte River Water Treatment Facility (WTP) at Ashland to the distribution system within the City. The distribution system consists of distribution mains, reservoirs, and pumping stations that deliver water from the transmission system to LWS's customers.

The transmission system pumps treated water from the high service pumping stations at the WTP through large diameter mains to two transmission facility sites, Northeast and 51st Street in the City, and the Low Service Level including "A" Street and Vine Street. From those sites and the Low Service Level, the water is either transferred to other transmission facility sites or is delivered to other service levels.

3.1 Transmission System

3.1.1 High Service Reservoirs and Pumping

The high service pumping stations take suction from two finished water underground storage reservoirs at the WTP with a total storage capacity of 9.0 million gallons (MG). The North Reservoir, with a capacity of 3.0 MG, and the South Reservoir, with a capacity of 6.0 MG, are supplied from the West Plant and the East Plant, respectively. The two reservoirs are connected with a reservoir transfer line.

The high service pumping at the WTP consists of twelve pumps with capacities ranging from 14.1 million gallons per day (MGD) to 21.6 MGD each. Some of the pumps have variable frequency drives (VFDs) and others are powered by diesel engines.

Up to 48 MGD of water can be pumped from the High Service Pumping Stations at the WTP directly to the Low Service Level. Under higher flow conditions, which result in greater head losses in the transmission mains, the water must be re-pumped into the Low Service Level by pumps located at the Northeast, 51st Street, and "A" Street locations. Under even greater flow rates, a transfer pump at the Northeast location is used to deliver flow to the 51st Street Reservoir, and transfer pumps at the 51st Street location are used to deliver flow to the "A" Street Reservoirs.

3.1.2 Ground Storage

A total of 79 MG of underground storage is provided on the transmission system from the WTP for pumping to the distribution system, as summarized in Table 3-1.

Table 3-1 Ground Storage Facilities

Reservoir	Total Capacity (MG)[1]
Platte River Water Treatment Facility Storage	
North Reservoir	3.0
South Reservoir	6.0
Lincoln Storage	
Northeast	10.0
51st Street	12.0
"A" Street	28.0
Vine Street	20.0
Total	**79.0**

3.1.3 Pumping

The transmission system pumping stations have pumps that supply water to other transmission system facilities as well as deliver water to the distribution system. The Northeast Pumping Station has a transfer pump that can supply water to the 51st Street Reservoirs. The 51st Street Pumping Station has transfer pumps that supply water to the "A" Street Reservoirs. The Northeast and 51st Street Pumping Stations can deliver water directly into the Low Service Level. The "A" Street Pumping Station can deliver water directly into the Low and High Service Levels.

3.2 Distribution System

The LWS service area is currently divided into six major service levels: Northwest, Belmont, Low, High, Southeast, and Cheney. In 2001, the Cheney Service Level was created in the southeast portion of the service area to serve new development on high ground. In 2002, the Northwest Service Level was created to serve a new development on high ground in that area.

3.2.1 Service Levels

Ground elevations within the existing service area range from about 1,124 feet (United States Geological Society [USGS] datum) along Salt Creek in the Low Service Level to about 1,429 feet in the Cheney Service Level. The highest ground is located in the northwest and southeast portions of the service area. Service level boundaries are established to maintain acceptable distribution system pressures. The service level boundaries should have sufficient flexibility to allow minor modifications using isolation or control valves to provide adequate service pressures, particularly at higher elevations and in developing areas.

The static hydraulic gradient for each of the six service levels is established by the maximum water service elevation of floating storage facilities within the service area or pressure reducing valve (PRV) settings. The ground elevations served and static hydraulic gradient for each service level are shown in Table 3-2.

Table 3-2 Service Levels

Service Level	Ground Elevation[1] (ft)	Static Hydraulic Gradient (ft)
Northwest	1,241 – 1,310	1,460[2]
Belmont	1,134 – 1,288	1,400[3]
Low	1,124 – 1,229	1,313[3]
High	1,152 – 1,314	1,420[3]
Southeast	1,225 – 1,282	1,500[3]
Cheney	1,335 – 1,429	1,580[3]

Notes:
1. *Based on ground elevations of meters within the service levels; USGS datum.*
2. *Currently established by PRV setting at pumping station discharge.*
3. *Established by overflow elevation of floating storage within service level.*

3.2.2 Storage Capacity Summary

A total of 52.5 MG of floating storage is provided in the distribution system. A summary of floating storage capacities by service level is presented in Table 3-3. All of the service levels have existing floating storage except for Northwest.

Table 3-3 Distribution System Floating Storage Facilities

Service Level	Reservoir	Total Capacity (MG)[1]
Belmont	Air Park	3.0
	Northwest 12th Street	4.5
	Total	**7.5**
Low	Vine Street	20.0
	Pioneers Park	4.0
	Total	**24.0**
High	Southeast	5.0
	South 56th Street	4.0
	Total	**9.0**
Southeast	Yankee Hill	10.0
Cheney	Cheney	2.0
Grand Total		**52.5**

3.2.3 Pumping Capacity Summary

Total and firm capacities for existing distribution system pumping stations are summarized in Table 3-4. Firm capacity is the total capacity of all pumps within a pumping station with the largest pump out of service.

Table 3-4 Distribution System Pumping Capacity

Service Level	Pumping Station	Number of Pumps	Installed Capacity (MGD)	Firm Capacity (MGD)
Northwest	Northwest 12th Street	5	9.5	6.3
Belmont	Belmont	4	30.2	21.1
	Merrill	2	7.4	3.7
	Pioneers	3	10.0	5.0
	Total		**47.6**	**29.9**
Low	Northeast[1]	5	85.7	65.5
	51st Street[1]	4	40.4	30.3
	"A" Street	3	28.6	18.2
	Total		**154.7**	**114.0**
High	"A" Street	4	36.4	27.3
	West Vine Street	4	75.7	55.5
	South 56th Street	3	13.5	9.0
	Total		**125.6**	**91.8**
Southeast	Southeast	2	20.2	10.1
	East Vine Street	4	30.4	21.3
	Total		**50.6**	**31.4**
Cheney	Cheney	5	9.3	6.2

Note:
1. Transfer pumps not included.

3.2.4 Northwest Service Level

The Northwest Service Level was placed into service in 2002 to serve new development on high ground in the northern portion of the City.

3.2.4.1 Northwest 12th Street Pumping Station

The Northwest 12th Street Pumping Station, also known as the Fallbrook Pumping Station, is a pre-packaged above-grade pumping station containing five pumps with a firm capacity of 6.3 MGD. The Northwest 12th Street Pumping Station pulls suction from the Northwest 12th Street Reservoir.

3.2.5 Belmont Service Level

The Belmont Service Level serves the northwest part of the City, including the Lincoln Municipal Airport. The Belmont Service Level is supplied by the Belmont, Merrill Street, and Pioneers Pumping Stations. The Belmont Service Level static hydraulic gradient of 1,400 feet is established by the overflow elevation of the Air Park and Northwest 12th Street Reservoirs.

3.2.5.1 Belmont Pumping Station

The Belmont Pumping Station pulls suction from the Low Service Level mains. It contains four pumps with a firm capacity of 21.1 MGD that boost from the Low Service Level to the Belmont Service Level.

3.2.5.2 Merrill Pumping Station

The Merrill Pumping Station contains two pumps with a firm capacity of 3.7 MGD that boost from the Low Service Level to the Belmont Service Level. City staff reports that the Merrill Pumping Station is not currently used since it is undersized for today's demand.

The Merrill Pumping Station site also has a standpipe-style surge tank that helps manage transient pressures within the 51st Street Pumping Station to "A" Street Reservoirs transfer main.

3.2.5.3 Pioneers Pumping Station

The Pioneers Pumping Station contains three pumps with a firm capacity of 5 MGD that boost from the Low Service Level to the Belmont Service Level. There is space for addition of a fourth pump in the station.

3.2.5.4 Air Park Reservoir

The Air Park Reservoir is a 3.0 MG reservoir that floats on the Belmont Service Level, has an overflow elevation of 1,400 feet and has a sidewater depth of 95 feet.

3.2.5.5 Northwest 12th Street Reservoir

The Northwest 12th Street Reservoir is a 4.5 MG reservoir that floats on the Belmont Service Level, has an overflow elevation of 1,400 feet, and has a sidewater depth of 75 feet.

3.2.6 Low Service Level

The Low Service Level serves the area bordering Salt Creek and encompasses the main business district, the University of Nebraska, and major industrial areas. The 51st Street, Northeast, and "A" Street Pumping Stations supply the Low Service Level. The Low Service Level is also served by the Vine Street Reservoir and the Pioneers Park Reservoir.

3.2.6.1 Northeast Pumping Station and Reservoir

The Northeast Reservoir has a storage volume of 10.0 MG. There are two 5.0 MG reservoirs which are supplied from the WTP. The reservoirs have an overflow elevation of 1,135 feet and a sidewater depth of 18 feet..

The Northeast Pumping Station contains one transfer pump, with a rated capacity of 31,250 gallons per minute (gpm) (45 MGD) at 60 feet. This transfer pump was replaced in 2007 and discharges to a transmission main which extends to the 51st Street Reservoir. A VFD allows the pumping capacity to vary from about 60- to 100-percent of the rated capacity at maximum speed (range of 16 MGD to 45 MGD).

The Northeast Pumping Station contains five Low Service Level distribution system pumps with a firm capacity of 65.5 MGD that boost to the Low Service Level. One pump is equipped with an eddy current drive and is not currently used due to heat generated by this drive.

3.2.6.2 51st Street Pumping Station and Reservoirs

The 51st Street Reservoirs include 6.0 MG, 5.0 MG, and 1.0 MG ground storage reservoirs. The 5.0 and 1.0 MG reservoirs have overflow elevations of 1,148 feet and sidewater depths of 14.2 feet. The 6.0 MG reservoir has an overflow elevation of 1,148 feet and a sidewater depth of 15.33 feet.

The pumping station contains three transfer pumps which pump to the “A” Street Reservoirs. The transfer pumps were replaced in 2001 with new units each with a rated capacity of 10,500 gpm (15.1 MGD) at 185 feet for a firm capacity of 30.2 MGD.

The 51st Street Pumping Station contains four Low Service Level distribution system pumps. New pumps and motors were installed in 2001 with the same rated capacity of the old units of 7,000 gpm (10.1 MGD) at 230 feet for a firm capacity of 30.3 MGD that boosts water to the Low Service Level.

3.2.6.3 “A” Street Pumping Station and Reservoirs

The “A” Street Reservoirs consist of five ground storage reservoirs that have a total capacity of 28.0 MG and are supplied from the 51st Street Pumping Station and from the Vine Street Reservoir. The five reservoirs have different overflow elevations. However, the reservoirs are interconnected and float together establishing a common hydraulic gradient.

The “A” Street Pumping Station, constructed in 1984, is a dual level pumping facility that discharges to the Low and High Service Levels. The station contains two Low Service Level pumps, each with a rated capacity of 6,300 gpm (9.1 MGD) at 155 feet. The station also contains two High Service Level Pumps, each with a rated capacity of 6,300 gpm (9.1 MGD) at 265 feet. Three satellite pumps are located at the “A” Street facilities in three separate buildings.

3.2.6.4 Vine Street Reservoir

The Vine Street Reservoir was expanded from 10.0 MG to 20.0 MG in 2001. The two 10 MG reservoirs float on the Low Service Level with an overflow elevation of 1,313 feet and a sidewater depth of 30 feet.

The reservoir provides suction storage for the adjacent Vine Street Pumping Station, which supplies the High and Southeast Service Levels. The reservoir can also be used as a supply to the “A” Street Reservoirs.

3.2.6.5 Pioneers Park Reservoir

The Pioneers Park Reservoir is a 4 MG reservoir that floats on the Low Service Level with an overflow elevation of 1,313 feet and a sidewater depth of 54 feet.

3.2.7 High Service Level

The High Service Level serves the areas south and southeast of the Low Service Level. It is supplied by the “A” Street and Vine Street Pumping Stations.

3.2.7.1 “A” Street Pumping Station

The “A” Street Pumping Station contains two High Service Level pumps, each with a rated capacity of 6,300 gpm (9.1 MGD) at 265 feet. The “A” Street facilities also contains two satellite pumping stations that discharge to the High Service Level each with a rated capacity of 6,300 gpm (9.1 MGD) at 250 feet. With the main station and satellite pumps, the High Service Level pumps have a firm capacity of 27.3 mgd.

3.2.7.2 Vine Street Pumping Stations

The High Service Level station at the Vine Street Pumping Stations, also called the West Vine Pumping Station, contains four pumps. One pump has a rated capacity of 10,500 gpm (15.1 MGD) at 115 feet and is equipped with an eddy current drive. The other three pumps have a rated capacity of 14,000 gpm (20.2 MGD) at 115 feet. The High Service Level pumps have a firm capacity of 55.5 MGD.

The Southeast Service Level station, also called the East Vine Pumping Station, was constructed in 2001 in conjunction with expansion of the Vine Street Reservoir. The station contains two pumps, each rated 7,000 gpm (10.1 MGD) at 210 feet for a firm capacity of 10.1 MGD to the Southeast Service Level. One VFD is located in the station but was taken out of service in 2012; the pumps are now operated with soft start contactors.

3.2.7.3 South 56th Street Reservoir and Pumping Station

The South 56th Street Reservoir is a 4.0 MG reservoir that floats on the High Service Level with an overflow elevation of 1,420 feet and a sidewater depth of 62 feet.

In 1998, a booster pumping station was added. The station contains three pumps each rated 3,125 gpm (4.5 MGD) at 50 feet for a firm capacity of 9.0 MGD.

3.2.7.4 Southeast Reservoir

The Southeast Reservoir is a 5.0 MG reservoir that floats on the High Service Level with an overflow elevation of 1,420 feet and a sidewater depth of 60 feet..

3.2.8 Southeast Service Level

The Southeast Service Level serves the high ground elevations in the southeastern section of the City. The Southeast Service Level is supplied by the Southeast Pumping Station and the Southeast Pumps at the Vine Street Reservoir and Pumping Stations site.

3.2.8.1 East Vine Street Pumping Station

The Southeast Service Level Pumping Station at Vine Street, also known as East Vine Street was constructed in conjunction with expansion of the Vine Street Reservoir. The East Vine Street Pumping Station contains two constant speed pumps each rated at 7,000 gpm (10.1 MGD) at 210 feet that pump to the Southeast Service Level.

3.2.8.2 Southeast Pumping Station

The Southeast Pumping Station contains four pumps, two rated at 4,200 gpm (6.1 MGD) and two rated at 6,300 gpm (9.1 gpm) at 155 feet for a firm capacity of 21.3 MGD.

3.2.8.3 Yankee Hill Reservoir

The Yankee Hill Reservoir a 10.0 MG reservoir that floats on the Southeast Service Level and has an overflow elevation of 1,500 feet and a sidewater depth of 75 feet.

3.2.9 Cheney Service Level

The Cheney Service Level was placed into service in 2001 to serve high ground in the southeast corner of the City. A portion of the existing Southeast Service Level was converted to the Cheney Service Level. The Cheney Pumping Station was installed in 2001. The Cheney Elevated Reservoir floats on the Cheney Service Level to create the static hydraulic gradient of 1,580 feet.

3.2.9.1 Cheney Pumping Station

The Cheney Pumping Station is a pre-packaged below-grade pumping station containing five pumps with a firm capacity of 6.2 MGD.

3.2.9.2 Cheney Elevated Reservoir

The Cheney Elevated Reservoir is a 2.0 MG reservoir that floats on the Cheney Service Level and has an overflow elevation of 1,580 feet and a sidewater depth of 40 feet.

3.2.10 PRV Station Summary

Currently, the City has five PRV installations. PRVs are used to reduce excessive pressures in a localized area or to connect two different pressure zones for exchange of water between them one direction or the other depending on the purpose of the PRV at the particular location.

3.3 Existing Storage and Pumping Operational Plan

LWS has two main operational plans for the water system: one for the summer demand season and one for the winter demand season. Two different plans are needed for the seasonal demand variations, primarily driven by outdoor water use during the summertime. During the peak summer months, demands can be in excess of 80 MGD, while winter demands can be less than 12.5 MGD. Changes in pumping scenarios and use of storage facilities are necessary in winter months to maintain adequate disinfectant residual and to help minimize water age in the system.

3.3.1 Summer Operations

Summer operations present a challenge to LWS not only due to increased demand for water but also due to the increased demand for electricity resulting in higher rate schedules during the SP period. In order to keep production energy costs at a minimum, LWS has entered into TOU agreements with both Lincoln Electric System (LES) and Omaha Public Power District (OPPD). The Lincoln Electric System (LES) TOU agreement affects the operation of the Northeast Pumping Station and the Omaha Pubic Power District (OPPD) TOU agreement affects operations at the WTP. If the actual power use exceeds the predetermined power consumption established in the TOU agreements, the power utility will increase the demand charge assessed to LWS. The increased demand charges remain in effect over the next year. Therefore, it is very costly for LWS to exceed the power demands outlined in the TOU agreements.

3.3.2 Winter Operations

Due to the decreased demand in the winter months, LWS staff adjusts water production operations during this period. The reduced demand affects both the transmission pumping operations and reservoir operations.

3.4 Reported Operational Issues and Needs

LWS has identified several areas in the distribution system that have operational challenges that were evaluated and addressed as part of this chapter.

3.4.1 Transmission System

LWS has requested an evaluation of when an additional WTP high service pump would be required and if additional pumps are required in the future.

At the Northeast Pumping Station, a pump is equipped with an eddy current drive and generates excessive heat. LWS would like to convert this pump to a constant speed pump removing the eddy current drive.

At the 51st Street Pumping Station and Reservoirs, there are some valves which LWS has identified as candidates for replacement. The current valves are manually operated and are at or near the end of their service life. LWS would like to automate the valves at this location to allow for remote operation.

3.4.2 Pumps and Pumping Stations

At the West Vine Street Pumping Station, a high service level pump currently has an eddy current drive, which can be inefficient at reduced speeds. At this time, the eddy current drive is no longer working and as a result the pump is out of operation. LWS would like to remove the eddy current drive and operate the pump as a constant speed pump. The Vine Street Pumping Station and other pumping stations have power factor issues that have been addressed.

Merrill Street Pumping Station has not been run in years and needs significant repairs to return it to an operable condition. This pump station is too small, and due to its condition and location in the system is a good candidate for decommissioning.

Northwest 12th Street (Fallbrook) and Cheney Pumping Stations are package pumping stations. These pumping stations are now over 10 years old and issues are starting to present themselves.

The South 56th Street Pumping Station that can boost pressures in the High Service Level has never operated. At this time the station needs maintenance in order to run. Since the pumping station has not and likely will not be run in the future, it is recommended to decommission this pumping station by removing the pumps and VFDs.

Belmont and Southeast Pumping Stations both have cavitation issues.

3.4.3 Distribution System

Various potential bottlenecks and low pressure areas were reported in the system near pumping stations during higher demand periods. Using the hydraulic model and estimated headloss under maximum hour conditions, these areas can be located and recommendations formed to alleviate the bottlenecks.

The Yankee Hill Road Main Improvements are expected to be constructed in mid-2014 and so an immediate capital improvement project has been included in the CIP. The Arbor Road Main Improvements are expected to be constructed when development occurs north of I-80 and when the I-80 Pumping Station is constructed.

A significant area of economic growth and redevelopment is the Nebraska Innovation Campus in the vicinity of North Antelope Valley Parkway and Salt Creek Roadway. Pre-design and preliminary drawings have been completed for this development. This area of the system is in the Belmont Service Level and has a couple of existing feeds and a redundant supply from Merrill Pumping Station. With the potential decommissioning of Merrill Pumping Station, a redundant supply including fire flows may be needed to serve this area in the future.

4.0 Master Planning Analyses

Hydraulic analyses were conducted to evaluate the performance of LWS's distribution system and to establish an improvement program to reinforce the existing system and allow expansion to meet projected water demands through the year 2060. Alternative improvements were investigated to identify those most effective in meeting future system needs. Criteria used to develop the improvement program include increasing system reliability, simplifying system operations, effectively utilizing system storage, and maintaining minimum pressures under the maximum hour demand (MHD) conditions. This section discusses development of the hydraulic computer model and results of the analyses performed.

Computer hydraulic analysis is a method of predicting hydraulic gradients, pressures, and flows across the water distribution network under a given set of conditions. The hydraulic gradient depends upon the magnitude and location of system demands, characteristics of the pipes in the distribution system, and the flows and gradients at network boundaries such as reservoirs and pumping stations. The headloss through each pipe is a function of flow rate, pipe diameter, length, and internal roughness. The available head or pressure, at any point in the network, is the difference between the hydraulic gradient and the ground elevation.

The distribution system model was run using Hazen-Williams' Equation for pressurized pipes to calculate headloss in the system:

$$h_L = 4.727C^{-1.852}d^{-4.871}Lq^{1.852}$$

where:

h_L = headloss, ft

C = Hazen-William's roughness coefficient

d = Pipe diameter, ft

L = Pipe Length, ft

q = Flow, cfs

4.1 Computer Model

The water distribution system was evaluated using the network analysis program, InfoWater by Innovyze, Version 10.0, Update 7, which operates in the Esri ArcMap environment. The modeling software can display ArcGIS layers that are exported from LWS and the City geographic information system (GIS) databases as the basis for the model and use as background information.

The physical characteristics of the water distribution system in the computer model include ground topography, reservoir elevations, pump characteristics, control valve characteristics, fire hydrant locations, and pipe diameter, length, and interior roughness. Historical and projected water demands are assigned to the computer model to represent the planning year conditions.

The computer model of the water distribution system was updated from the 2007 water model using current GIS data provided by LWS. The model includes all service levels and the transmission system from the WTP at Ashland to the City's distribution system in a single model. All pumping stations which discharge to the distribution system or boost water from one service level to another were incorporated into the model.

The water system network GIS provided by LWS in July 2013 was used as the basis for updating the hydraulic model. The GIS water mains layer contains all known existing pipelines and near-term proposed pipelines. Additional as-constructed drawings were provided for some areas of the system under construction or recently built that may have not been contained in the GIS water mains layer.

A one-half mile future water main grid was used in the model for future development areas within the LPlan 2040 boundary and a one mile water main grid was used outside of the LPlan 2040 and inside the LPlan 2060 boundaries. No future mains were modeled outside of the LPlan 2060 boundary unless to keep them on the one mile main grid.

The model contains all water mains of 4 inches in diameter and greater, including the transmission mains, public and private distribution mains, and hydrant laterals. Service mains were not included in the model. The model is close to what is referred to as an "all-pipes model" since it represents most of the pipes within the source GIS.

4.1.1 Pipe Friction Coefficient (C-value)

The pipe friction coefficient, "C" value in the Hazen-Williams empirical equation for pipe flow presented above, is an index of pipe hydraulic capacity due to internal roughness. The "C" value is dependent upon a number of factors including pipe material, type of lining, pipe age, cross-sectional area, amount of tuberculation, and thickness of calcium carbonate deposits. High "C" values represent smoother interior surfaces while low "C" values represent rough or reduced diameter interior surfaces. The typical "C" value for a new cement-lined ductile iron pipe is about 130, and decreases as pipes age. Prior to the 1960s mains were generally not lined with cement mortar, resulting in tuberculation and lower "C" values.

The mains in the City's water distribution system are mostly lined cast or ductile iron. The "C" values assigned in the 2007 computer model were maintained for the most part in the updated model. "C" values ranged from 120 for newer large diameter transmission mains, to 80 for older and smaller mains in the distribution system. A few existing "C" values were modified as part of the model validation process to reach a closer match between observed operations data and the model results. All future mains were modeled with a "C" value of 130.

4.1.2 Demand Allocation

An updated demand allocation was conducted to replace the 2007 model demand allocation. The InfoWater computer model enables allocation of demands to junctions based on up to ten classes or fields. Base year (2012) average day demands (ADDs) were allocated to the computer model using existing metered sales data and water capacity requirements including non-revenue water as described in *Chapter 2*. The base year allocation uses the first 3 demand fields (designated as existing residential, non-residential, and non-revenue demands). Future year demand allocation was based on transportation analysis zone (TAZ) data and the water capacity requirement values discussed in *Chapter 2*. The future year allocation uses the next 3 demand fields (designated as future residential, non-residential, and non-revenue demands) and is cumulative for each planning year.

The majority of development areas resulting in future demands are at the system extents; however, about 10 percent is expected to be infill development. This is addressed in the demand allocation process by applying the additional future demand using the TAZ population increases with corresponding non-residential sales to the existing system areas. The land use is not expected to change drastically within the infill development areas to change fire flow requirements.

4.1.2.1 Base Year Demand Allocation

The base year model scenario is based on 2012 data, the last full year of demand data from the water system. Base year ADDs of 38.4 MGD established in *Chapter 2* were allocated to the hydraulic model of the existing distribution system. LWS provided geocoded metered sales information for every account during fiscal years (FY) 2011/2012 and 2012/2013. The information included meter identification (ID), account number, account address, service address, bi-monthly metered units in 100 cubic feet units (equal to 748 gallons), and a user classification code (residential or non-residential). The account number identified the meter cycle for each account. The data consisted of a total of 82,800 records.

Initially, the GIS meter locations were matched to the meter data records using the meter IDs. Due to differences between GIS locations and the billing data records, there is not a one-to-one match between the two data sources. To match additional meter sales between the billing meter data and the GIS locations, the following steps were taken:

1. Matching GIS meter service addresses to billing addresses.
2. Matching GIS meter account numbers to billing account numbers.
3. Matching significant users (over 10,000 gpd) in the billing records to GIS meters by manually mapping billing addresses to determine if they were likely service addresses and assigning it to the nearest GIS meter.

The process resulted in a 97 percent match in total meter locations (80,688 of 82,821) and in total water sales volume (35.2 of 36.2 MGD) which was determined to be acceptable based on experience with other planning efforts and a similar analysis conducted in the 2007 Facilities Master Plan (2007 Master Plan).

This metered sales data by class was used to determine the spatial distribution of the total demand. This allocation method precisely reflects the actual distribution of metered water sales in Year 2012. Non-revenue water, approximately 10.2 percent system-wide, for the base year was evenly allocated to each model node based on the total calculated non-revenue volume by service level at each node to arrive at the total system ADD of 38.4 MGD.

4.1.2.2 Year 2025, 2040, and 2060 Demand Allocation

Future demands were allocated to the model based on population and commercial/industrial area data for each TAZ, as provided by the Lincoln-Lancaster County Planning Department. Where a TAZ boundary was split by a service level boundary, the TAZ was split (TAZ segment) and the population was allocated to each TAZ segment based on the area split. Service level specific per-capita water use rates were then applied to the population to produce residential sales for every TAZ segment. Ratios for the breakdown of residential to non-residential sales by service level as described in *Chapter 2* were used to determine the total sales for each TAZ segment. TAZ segments received a weighted portion of sales from the overall non-residential demands in the service level by the presence and increase of commercial/industrial area within the TAZ segment. The base year per-capita water sales rates used to calculate future residential demands are shown in Table 4-1. The average base year system wide per-capita water sales rate is 86 gallons per capita per day (gpcd).

Table 4-1 Service Level Per-capita Residential Sales Rates

Service Level	Per-capita Residential Sales Rate (gpcd)
Northwest	130
Belmont	78
Low	60
High	90
Southeast	125
Cheney	175

The thiessen polygon method was used to allocate the sales by TAZ to model nodes. A thiessen polygon represents boundaries that define the area that is closest to each point relative to all other points. The total sales contained by the TAZ area for each respective node was allocated to that node. Thiessen polygons were bounded by service level boundary so that a node in one service level would not capture demands represented by a TAZ segment in a different service level.

Future non-revenue water volume, approximately 6.7 percent system-wide, was allocated evenly to each model node by service level. Non-revenue water demands were allocated to nodes with residential or non-residential demands at a value equal to 6.7 percent of the total demand.

4.1.3 Model Validation

The day that was selected for use in the model validation was July 24, 2012, the day of maximum day demand (MDD) for that year. The MHD was also used from that same day at 6 am for validation. The simulation was performed and slight changes were iteratively made until the results closely resembled the Supervisory Control and Data Acquisition (SCADA) data provided. With the exception of the Belmont Service Level pumping station flows, the model is producing a good match system-wide including both the distribution and transmission systems. For the purposes of this Master Plan, the model is producing good system-wide results and will provide the analysis tool needed to plan for the future of the water system.

4.1.4 Model Scenarios

The following scenarios were developed in the hydraulic model in support of the distribution system hydraulic evaluations:

- Base Year (2012)
  - Maximum Day
  - Maximum Hour
- Year 2025
  - Maximum Day
  - Maximum Hour
- Year 2040
  - Maximum Day
  - Maximum Hour
- Year 2060
  - Maximum Day
  - Maximum Hour

4.2 Evaluation Criteria

Maximum day and maximum hour evaluation criteria are established for analyzing the performance of the distribution system under existing and future conditions. The results of planning year analyses compared against the evaluation criteria are the basis for required improvements and their sizing. Two criteria tiers have been established to two levels of performance deficiencies within the system: critical and moderate. Water age evaluation criteria are established in Section 5.0.

4.2.1 Maximum Day Conditions

The MDD conditions are evaluated against the following criteria:

- **Required Fire Flow Availability**
    - Tier 1
        - 50 to 100 percent
    - Tier 2
        - Less than 50 percent
- **Pumping Station Capacity**
    - Tier 1
        - Below firm capacity by 1 pump
    - Tier 2
        - At firm capacity

4.2.2 Maximum Hour Conditions

The MHD conditions are evaluated against the following criteria:

- **Distribution Pressure**
    - Tier 1
        - Low: 30 to 40 pounds per square inch (psi)
        - High: 110 to 120 psi
    - Tier 2
        - Low: Less than 30 psi
        - High: Greater than 120 psi
- **Distribution Headloss**
    - Tier 1
        - 3 to 5 feet/1,000 feet
    - Tier 2
        - Greater than 5 feet/1,000 feet
- **Storage Utilization**
    - Tier 1
        - 90 to 100 percent
    - Tier 2
        - Greater than 100 percent

4.2.3 Storage Replenishment Conditions

The storage replenishment conditions for each planning year are evaluated against the following criteria:

- **Storage Refill Capability**
  - Tier 1
    - 125 to 100 percent
  - Tier 2
    - Less than 100 percent

4.2.4 Average Month and Minimum Month Conditions

The average month demand (AMD) and minimum month demand (MMD) conditions are evaluated against the following criteria:

- **Water Age**
  - Tier 1
    - 10 days to 15 days
  - Tier 2
    - Greater than 15 days

4.3 Hydraulic Analyses

A series of flow balance analyses were conducted under the four planning year demand conditions including the base year (2012). Storage and pumping deficiencies were evaluated in conjunction with improvements relating to the LWS capital improvement program (CIP).

Maximum day and maximum hour steady state hydraulic simulations using the model were performed in order to verify the transmission and distribution response under varying demand conditions compared against the evaluation criteria. Pumping and storage were evaluated on a service level-basis to determine existing and future needs to supply and store adequate water.

4.3.1 Base Year (Immediate)

A series of analyses were conducted under the base year (2012) demand conditions, and included main improvements required to serve the Tier I – Priority A limits. The immediate improvements necessary to correct existing deficiencies are categorized as immediate (2014-2019) recommended improvements.

4.3.1.1 Transmission

It was determined that the transmission system has enough capacity in the next six years to adequately supply the distribution system. However, due to excessive heat generated by the eddy current drive at the Northeast Pumping Station, the pump should be converted to a constant speed pump by 2016.

4.3.1.2 Storage

No storage improvements have been recommended as immediate improvements.

4.3.1.3 Pumping

Due to its non-operational condition, the eddy current drive should be removed from West Vine Street and the pump converted over to a constant speed pump by 2014. This will allow this pump to be operated again and restore the existing West Vine Street Pumping Station capacity.

The South 56th Street Pumping Station which remains unused should be assessed to evaluate its potential operational value. Based on the modeling results, the need for this facility is questionable and as a result it is recommended for decommissioning.

The existing Northwest 12th Street and Cheney Pumping Stations have adequate capacity but are nearing the end of their useful life as they were intended as temporary pumping stations. A permanent Yankee Hill Pumping Station by 2017 with 12.0 MGD firm / 18.0 MGD installed capacity is recommended to be installed. With this increase in pumping capacity to Cheney Service Level, Cheney Pumping Station is recommended for decommissioning by 2019. A permanent Northwest 12th Street Pumping Station by 2019 with 8.0 MGD firm / 12.0 MGD installed capacity is recommended. The existing Northwest 12th Street Pumping Station can be decommissioned after the new one is constructed by 2020.

A booster pumping station at I-80 with 3.0 MGD firm / 6.0 MGD installed capacity is recommended to be constructed by 2018, depending on actual development timing, and would be planned to be decommissioned by 2060. By 2060, the expanded section of Belmont will be connected to the rest of the system. The booster pumping station should be constructed with the first development to take place in the area served. The station should have a firm capacity of 3.0 MGD with an additional fire pump and should be constructed west of 56th Street at I-80. If there is one primary private developer for this area, they could construct the pumping station to be handed over to LWS upon completion, as long as they follow LWS's design and performance standards.

Replacing a pump at the East Vine Street Pumping Station with a 20.2 MGD pump for the Southeast Service Level is recommended by 2019.

Due to its condition and lack of use, Merrill Street Pumping Station is recommended for decommissioning by 2020.

4.3.1.4 Distribution

The Yankee Hill Road Main Improvements should be constructed by the end of 2014 to provide redundancy and reduced headloss in its area of the distribution system.

Improvement mains are required to provide service to the development area located north of I-80. This area should be provided service at pressures equivalent to the Belmont Service Level. Additional improvement mains will be necessary to expand the system boundaries as development occurs.

4.3.2 Year 2025 (Short-Term)

A series of analyses were conducted under planning year 2025 demand conditions, and included main improvements required to serve the Tier I – Priorities A and B service limits. The improvements necessary to correct projected deficiencies are categorized as short-term (2020-2025) recommended improvements.

4.3.2.1 Transmission

By 2025, a transmission main from Northeast Reservoir and Pumping Station to Vine Street Reservoirs is recommended. This pipeline will allow for additional transmission system delivery flexibility including bypassing the Northeast Reservoir and Pumping Station with delivery straight from the WTP to the Low Service Level, Vine Street Reservoirs and on to "A" Street Reservoirs.

An additional high service pump rated at 20.9 MGD at 350 feet with a VFD is recommended to be added at the WTP. A predesign project should confirm pump flow and head requirements for service directly to the Low Service Level as well as to the Northeast Reservoir for the most efficient operation under both conditions. A properly designed pump and VFD should allow the flexibility for pumping to these two delivery points. There may be a slight energy savings in pumping directly to the Low Service Level from the WTP instead of double pumping at the Northeast Pumping Station.

4.3.2.2 Storage

The Northwest Service Level currently has no floating storage. For improved reliability and more balanced pumping operations, a 2 MG elevated storage reservoir and pipeline is recommended.

To meet a short-term storage deficiency in the High Service Level and support growth in the east portion of the City, Adams Reservoir, a 5 MG above-grade storage reservoir, is recommended by 2025.

4.3.2.3 Pumping

No pumping improvements have been recommended as short-term improvements.

4.3.2.4 Distribution

Several improvement main projects are recommended to support growth and address system bottlenecks under Year 2025 demands.

These improvements include:

- A water main on West 56th Street
- A transmission main from Vine Street Pumping Station to High Service Level
- A transmission main from High Service Level to Future Adams Reservoir

Additional improvement mains will be necessary to expand the system boundaries as development occurs.

A PRV station is recommended to feed water back from the Belmont Service Level to the Low Service Level during high demand periods and for improving local fire flow availability.

4.3.3 Year 2040 (Mid-Term)

A series of analyses were conducted under planning year 2040 demands, and included main improvements required to serve the Tier I – Priorities A, B, and C service limits. The improvements necessary to correct projected deficiencies are categorized as mid-term (2026-2040) recommended improvements.

4.3.3.1 Transmission

The completion of the transmission main between Northeast Reservoir and Vine Street Reservoir will create a reduction in the required total discharge head directly from the WTP to the Low Service Level which allows the WTP pumps to operate at a higher flow point on their curves. Therefore, the actual WTP high service pumping capacity at the WTP would be greater than the theoretical firm capacity. By year 2040, the demand will likely be high enough that one of the existing high service pumps would need to be replaced a pump with a rated capacity of 20.9 MGD and a VFD to allow for the flexibility to pump directly to the Low Service Level as well as to the Northeast Reservoir.

To serve High Service Level and downstream service levels, a new pumping station, Northeast II, consisting of three 20.2 MGD pumps for a 40.4 MGD firm capacity, is recommended to be installed at Northeast Reservoir site.

To support the distribution from the existing WTP and future Missouri River Project throughout the system, an additional parallel transfer main from Vine Street to “A” Street is recommended. A predesign evaluation should be conducted to evaluate the benefit and cost of installing this parallel transfer main compared against the continued delivery through and condition of the existing main.

Due to condition and the mid-term transmission capacity needed, one of the existing transmission mains from the WTP to “A” Street Reservoirs is recommended to be replaced with a larger transmission main by 2040.

4.3.3.2 Storage

A storage deficiency in the Cheney Service Level has been identified so an additional Cheney II Reservoir, a 3 MG elevated storage reservoir, and required pipeline is recommended to be installed by 2040. Additional storage may be required in this service level and should be supported by actual development as well as future studies and demand projections.

4.3.3.3 Pumping

By 2040, a 5.0 MGD additional pump is recommended to be added to Pioneers Pumping Station to help meet additional demands in the Belmont Service Level. This pump would fill the last available slot in the Pioneers Pumping Station.

An additional 20.2 MGD pump at the East Vine Street Pumping Station to Southeast Service Level is recommended by 2040. This pump would fill the last available slot in the East Vine Street Pumping Station.

4.3.3.4 Distribution

Hydraulic analyses show that the Belmont Service Level reservoirs currently operate at approximately equivalent gradients. However, as the demands within the Belmont Service Level increase, the difficulty of maintaining the same gradient at the two existing storage facilities will increase.

Currently, the Northwest 12th Street Reservoir has a valve that is ¼ closed. The total head loss caused by this valve setting is minimal because of the relatively low velocities that occur in the inlet pipe and the relatively small minor losses associated with such a setting. As demands increase, the flexibility of alternating pumping capacities between the Belmont Pumping Station and the Pioneers Pumping Station will be reduced as the pumping station maximum capacities are approached and both stations operate at or near maximum capacity. It may be necessary as demands increase to change the setting on this valve. LWS should consider the possibility of utilizing an existing motorized control. While it is understood that the utilization of this motorized control valve may add one more complication to the distribution system from an operations standpoint, it also contributes flexibility in control that may be useful during future peak demands. Northwest 12th Street reservoir also has an altitude valve, which can be programmed to open/close at various level set points that are not currently used.

Hydraulic analyses indicate that the planned capacity of the Pioneers Pumping Station must not exceed the delivery capacity within the Low Service Level to the extent that it becomes difficult to maintain water levels in the nearby Pioneers Park Reservoir. Under year 2040 maximum day conditions, hydraulic analyses determined that the existing pumping and transmission capacity in the Low Service Level could support maximum day flows up to about 10 MGD at the pumping station. Without additional significant main improvements in the Low Service Level, the maximum firm pumping station capacity should be limited to 10 MGD. The Pioneers Pumping Station could typically deliver about 3.0 to 4.0 MGD under current maximum day conditions. The lower magnitude of transfer from Pioneers Pumping Station contributes to high water age in the Pioneers Reservoir.

As demands increase past the year 2040 maximum day demands, the total firm pumping capacity into the Belmont Service Level will be surpassed and the South Belmont Pumping Station is recommended.

A connector main in the Belmont Service Level is recommended to connect the isolated area served by the I-80 and North 56th Booster Station to the main Belmont Service Level.

Additional improvement mains will be necessary to expand the system boundaries as development occurs.

4.3.4 Year 2060 (Long-Term)

A series of analyses were conducted under planning year 2060 demands, and included improvements required to serve the Tier II service limits. The improvements necessary to correct projected deficiencies are categorized as long-term (2041-2060) recommended improvements.

4.3.4.1 Transmission

With the potential completion of the Missouri River Project, a transmission main to the Vine Street Reservoir is recommended by 2060. This transmission main will deliver from 20.0 to 50.0 MGD of supply into the Vine Street Reservoir for distribution into the system.

Due to its eventual condition degradation, rehabilitation or replacement of the existing transmission main from the WTP to Vine Street Reservoir is recommended by 2060.

4.3.4.2 Storage

Due to increased demands in the Northwest Service Level, an additional elevated reservoir of 2 to 3 MG and pipeline may be required to be installed by 2060. Any additional storage required in this service level should be supported by actual development as well as future studies and demand projections.

Due to increased demands in the Belmont Service Level and the location of growth, Southwest Reservoir, an above-grade reservoir of 4 MG and pipeline, is recommended to be installed by 2060. The future Southwest Reservoir modeled in the 2040 analyses should have an overflow elevation of 1,400, similar to the existing storage facilities. During lower demand conditions this reservoir will operate at gradients close to the other two facilities. However, during high demand periods, it may be difficult to maintain the level in the future reservoir. Under these conditions the reservoir would be allowed to drop to normal levels of about 5 to 15 feet below the levels in the other Belmont Service Level reservoirs. Addition of the South Belmont Pumping Station by 2060 should help with this issue by boosting the hydraulic grade in the southern area of the Belmont Service Level. Any additional storage required in this service level should be supported by actual development as well as future studies and demand projections.

The expected increase in demand drives the installation for additional storage capacity in the High Service Level. To meet this need a Saltillo Road Reservoir of 3 MG above-grade storage and pipeline and an Adams Street Reservoir II of 5 MG above-grade storage and pipeline are recommended to be installed by 2060. Any additional storage required in this service level should be supported by actual development as well as future studies and demand projections.

To meet a mid-term and larger long-term storage deficiency in the Southeast Service Level, Rokeby Reservoir, a 5 MG above-grade reservoir, and pipeline is recommended to be installed by 2060. A total deficiency by 2060 was calculated to be 10 MG; however, due to the uncertainty of growth in this area of the City, it is recommended to approach storage in stages, as necessary, for the Southeast Service Level past the mid-term planning period. Any additional storage required in this service level should be supported by actual development as well as future studies and demand projections.

4.3.4.3 Pumping

At West Vine Street Pumping Station, an additional 20.2 MGD pump to the High Service Level is recommended to be installed by 2060. This pump would fill last available slot at this pumping station.

New growth and increased demands in the Belmont Service Level are served by a new pumping station, South Belmont, of 5.0 MGD firm and 8.0 MGD installed capacity by 2060. This new pump station will also replace the I-80 and 56th Street Booster Station capacity for the service level, which could allow for its decommissioning.

4.3.4.4 Distribution

Additional improvement mains will be necessary to expand the system boundaries as development occurs.

4.4 Fire Flow Analyses

In addition to supplying water for domestic, commercial, and industrial uses, a municipal distribution system should be capable of supplying an adequate and dependable flow for fire fighting. Although the annual volume of water used for fire fighting is relatively small, the rate of use may be quite high during fires. These high rates may impose critical demands on transmission, pumping, and storage facilities.

The base year maximum day demands were used to analyze potential fire flow deficiencies in the distribution system. Zoning-based fire flow requirements were established as a general indication of areas where potential deficiencies may be. The hydraulic model was run to determine available fire flow at each hydrant tee in the system and compared against the fire flow requirements.

Improvements to correct any deficiencies should be confirmed with additional hydraulic modeling in each project area to determine upsizing and looping requirements to ensure proper fire flow is made available. Also, improvements should be coordinated with on-going condition and main break evaluations to replace poor condition pipe with new upsized pipe to address both issues at the same time. Additional discussion on this is included in *Chapter 6 - Water Main Replacement Program*.

Detailed fire flow evaluations were performed and the results were incorporated into the development of the improvements program.

5.0 Water Age Analyses

Water age modeling was performed for the distribution system to identify areas in the distribution system that with high residence times. It is acceptable industry practice to use distribution system water ages as a surrogate indicator for many water quality parameters including disinfection by-product formation, disinfectant decay, corrosion control effectiveness, microbial re-growth, nitrification, and taste and odor issues. Water age should not be considered as the ultimate indicator of these aforementioned water quality characteristics, but in conjunction with other factors such as pipe characteristics, disinfection processes, distribution system operations, and water-use habits. However, water age can be quite useful in identifying distribution system deficiencies in terms of water quality.

5.1 Existing Water Quality Management

Pioneers, Air Park, Northwest 12th Street, South 56th Street, and Cheney Reservoirs are thought to have water age issues during certain times of the year. While the LWS does not conduct dedicated routine flushing of the water distribution system, each fire hydrant is inspected on an annual basis. When poor water quality is observed during the hydrant inspections, the hydrant will be opened and flushed until clear water is observed.

LWS has two locations within the water distribution system where the water mains are automatically flushed to address persistent low chlorine residuals. One location is in Air Park in the Belmont Service Level and the other location is in Waterford Estates in the High Service Level. The Waterford Estates location was noted as possibly being removed during this study and chlorine residuals to be monitored.

During the winter, measures are taken to reduce water age in seven of the reservoirs and maintain chlorine residuals. Low pressure alarms while trying to cycle the reservoirs to a lower level can be overridden and subsequently tank refill should be initiated.

5.2 Water Age Observations

Five extended period simulation scenarios were run against the base year AMD and MMD using the validated hydraulic model to determine what improvements could be made to the system to improve water age.

5.2.1 Model Results Observations

General observations from the water age modeling results are documented in the following paragraphs.

5.2.1.1 Overall System Age

Each water age scenario was simulated for a duration of 30 days (720 hours). Areas of the system fed by the Air Park, Southeast and Cheney tanks have the highest water age in the system. The overall system water age is 148 hours and 103 hours during MMD and AMD, respectively.

From the MMD alternative with less storage in the transmission system shows a large improvement to the age in the Southeast and Cheney Service Levels.

The AMD alternatives include two new tanks within the proposed Tiers I and II improvements. The average age in these scenarios show that the age will increase in the system caused by the extra future storage. The second average month alternative shows a small decrease in the overall age by moving water out of the Northeast Reservoir to the Vine Street Reservoir, but some of the tanks show an increase in age.

5.2.1.2 Reduced Transmission Storage

When the transmission storage is reduced by reducing the storage in half at the Northeast and 51st Street Reservoirs and reducing “A” Street Reservoir storage from 28 to 12 MG, the average age of the system is reduced from 148 to 125 hours. The largest impact can be seen in the Cheney and Southeast Service Levels. The age of water in the Yankee Hill Reservoir was reduced from 289 to 261 hours and in the Cheney Reservoir from 363 to 318 hours. Because the transmission reservoirs can only feed the distribution by pumping, the reduced storage will not impact the required storage for fire protection or pressures in the distribution system. Also, in the case of a winter storm or the loss of power and the system can no longer be fed by pumps, the transmission storage cannot be used to feed the system, so reducing the storage will not impact the ability to provide water in the case of power loss. Due to the amount of time to take these storage reservoirs out of service in the low demand season and to reinstate them in the spring may not be desired to obtain a 16 percent reduction in water age. In addition, the risk of losing source and emergency storage volume likely does not offset the 16 percent improvement in water age.

5.2.1.3 Transmission Pumping Modifications

The low flows supplying the system from the WTP causes an increase in age at the transmission storage facilities due to the low velocities in the large transmission mains. By allowing Northeast Reservoir to drain more and then quickly refilling it, the transmission mains will achieve a higher velocity and the water will reach the system faster than if constant low flows are maintained.

5.2.1.4 Air Park and Northwest 12th Street Reservoirs

Reducing the fill and drain levels of the Air Park and Northwest 12th Street Reservoirs will allow the reservoirs to cycle faster and more often and reduce the storage in the Belmont Service Level. The drain level should be brought all the way down to the low level alarm for the reservoirs. At this level with low headloss in the distributions system from low flows, the pressures will remain high enough to serve the distribution system. This should improve the overall quality of water within the Belmont Service Level as well as the Northwest Service Level.

5.2.1.5 Pioneers Reservoir and Control Valve

A control valve should be considered to provide for better control of the Pioneers Park Reservoir to allow for better fluctuation of water levels and reduced water aging. Under current conditions, the water level in the reservoir fluctuates very little resulting in high water ages and low chlorine levels in the area. The control valve would be located at the Pioneers Pumping Station. With the valve closed, the pumping station will draw down the reservoir. At a predetermined level in the reservoir, the valve would open allowing the reservoir to fill. The valve should be set-up to prevent high flow rates into the reservoir during the fill cycle resulting in possible low pressures in the Low Service Level. The reservoir fill can be controlled by equipping the valve with an upstream pressure sustaining set-point. The valve should be monitored closely and manual override of the valve should be available at the WTP control center.

Field testing including closing valves to simulate the proposed control valve operation is recommended prior to implementation of this project to determine actual behavior and any impact to water quality. The final solution could change or be deemed not beneficial based on field testing results.

5.2.2 In-Reservoir Water Management

Managing in-reservoir water is important; if there is poor mixing inside a reservoir, it can develop stagnant zones within the tank which can promote biological growth resulting in increased chlorine decay. This can exacerbate water quality issues within the service level by increasing average water age and decreasing chlorine residuals. An in-depth tank mixing study and improvements project should be incorporated into the CIP to evaluate adding mixers to selected tanks. During this study, computational fluid dynamics (CFD) modeling can be used to help find stagnant zones inside the storage reservoirs and determine improvements that would provide for more complete tank mixing. Increasing the operational storage (high to low operating levels) can also improve water age within storage in the system. However, doing this effectively reduces service pressure available and emergency storage volumes, so any changes should be eased into if further operating level reductions are agreed upon.

5.2.3 Automatic Flushing Hydrants

For each planning period, a minimum of three additional automatic flushing hydrants are recommended. In order to minimize the amount of water flushed from the system and remain in line with conservation efforts, an automatic flushing hydrant equipped with a sampling station is recommended. The automatic flushing hydrant can be programmed to only flush when the chlorine residuals get below a specified level, so it will only flush when it is needed. These hydrants will flush older water out of the system and bring newer water into areas with higher chlorine residuals. SCADA implementation is possible with some models of automatic flushing units to allow even further operator monitoring and control over flushing water from the system. In addition, flow rates from the flushing units could be modeled to show how water ages would be affected in the area of installation. This was not completed as part of this study but is recommended prior to their installation to determine potential effectiveness before capital expenditure is made.

Capital improvement projects for the immediate, short-term, and mid-term planning periods have been included in the capital improvement program. Costs for these flushing hydrants include a sampling station but no SCADA infrastructure for remote monitoring.

5.3 Water Age Conclusions and Recommendations

Conclusions and recommendations drawn from the distribution system water age analyses are summarized below:

- The seasonal reduction of storage volumes at all of the transmission storage facilities will reduce the water age throughout the distribution system. However, this is not recommended due to loss of key source and emergency storage and the problem will lessen itself over time as the demand in the system increases.
- Reducing the operational fill and drain levels of reservoirs will reduce the storage in the system and reduce the water age of the system. The only reservoir that does not have a lower winter operations level already is the Cheney Elevated Reservoir. It is recommended to reduce the winter lower operating level to 20 feet to help improve water age in the Cheney Elevated Reservoir. Increasing the operational storage (high to low operating levels) at other reservoirs can also improve water age. However, doing this effectively reduces service pressure available and emergency storage volumes, so any changes should be eased into if further operating level reductions are agreed upon.
- The Pioneers Park Control Valve or a similar water quality improvement would moderately improve water ages in the Pioneers Park Reservoir but has a negligible impact on water ages in the Belmont Service Level. Field testing is recommended including closing valves for testing the actual impact in the system should be conducted prior to improvement project implementation.
- The minimum water level drawdown in the Pioneers Park Reservoir is limited by distribution ability to refill it adequately appears to be close to 40 feet.
- An in-depth tank mixing study and improvements project should be incorporated into the CIP to evaluate adding mixers selected tanks. During this study, CFD modeling can be used to help find stagnant zones inside the storage reservoirs and determine improvements that would provide for more complete tank mixing.
- Automatic flushing hydrants with chlorine residual sampling functionality should be added in areas of low chlorine residuals to help discharge old water and bring in new to those areas of the system.
- Unidirectional flushing should be used instead of conventional flushing whenever possible to achieve higher velocities for more effective pipe cleaning. Unidirectional flushing also helps support valve exercising due to the use of isolation valves for maximizing flushing velocities in specific areas of the system.

- Consider building a booster chlorine station into the future Yankee Hill Pumping Station and a PRV that feeds from Cheney Service Level back into Southwest Service Level to improve chlorine residuals in the area of high water age. Costs have been included in the future Yankee Hill Pumping Station cost estimate for a booster chlorine station.
- Consider building a booster chlorine station into the future Northwest 12th Street Pumping Station and a PRV that feeds from Northwest Service Level back into Belmont Service Level to improve chlorine residuals in the area of high water age. Costs have been included in the future Northwest 12th Street Pumping Station cost estimate for a booster chlorine station.

Prior to implementation of any of the recommendations, an in-depth water age and quality study for each improvement project should be conducted to verify and optimize water age reducing measures and water quality enhancing measures and their effect throughout the distribution system. Energy use, conservation impacts, customer perception, emergency and risk management, cost-benefit and other related factors should be considered prior to implementation of any of these improvements.

Water quality complaints should be tracked in GIS with address, date, time of day, outside temperature, and latest chlorine residual level in the vicinity. This information can be used to determine which improvement type, in addition to temporary hydrant flushing, should be implemented first. Improvements should be focused on correcting the water quality worse areas first for the least cost possible as water quality is not a distribution system-wide issue. Overall, the recommended water quality improvements from this study are a small percentage of the overall capital improvement program.

6.0 Operational and Efficiency Analyses

This section covers a number of requested ancillary evaluations and improvement recommendations for operations and efficiencies including energy savings of existing system facilities.

6.1 Transmission System

Generally, to increase the efficiency and simplify the operations of the transmission system, as many pumping stations and reservoirs should be bypassed as possible during the range of demand conditions throughout the year. After automation of the 51st Street valves, the Northeast and 51st Street Pumping Station should be bypassed with flow directly to the “A” Street Reservoir. Actual system demands will dictate how much the “A” Street Pump Station needs to be used as it helps boost pressures in its area of the Low Service Level as it provides an alternate flow path to customers. These general transmission system objectives help reduce energy usage and improve water age.

6.1.1 Additional High Service Pumping at the WTP

Additional WTP High Service Pumping will be required as growth occurs. A new pump, with a rated capacity of 20.9 MGD and rated head of 350 feet, should be installed by year 2025. The addition of this pump will fill all existing high service pumping bays at the WTP. One additional pump rated at 20.9 MGD and 350 feet for head should be installed by the year 2040. This pump should replace the existing pump which has lower discharge head characteristics than the other high service pumps. Depending on where the long-range water supply will come from, additional pumps and pumping station at the WTP may be required by 2060.

A predesign project should proceed with all pump design projects to confirm required pump flow and head requirements for service directly to the Low Service Level as well as to the Northeast Reservoir for the most efficient operation under both conditions. A properly designed pump and VFD should allow the flexibility for pumping to these two delivery points. There may be a slight energy savings in pumping directly to the Low Service Level from the WTP instead of double pumping at the Northeast Pumping Station.

6.1.2 Northeast and 51st Street Pumping Stations Energy Management

LWS is interested in the feasibility of taking the Northeast and 51st Street Pumping Stations out of operation during the TOU period which is 2 pm to 8 pm Monday through Friday. This is approximately an $8.25 to $18 increase in demand charge per kilowatt. Currently, the Northeast Pumping Station is taken out of service during the TOU period, but not both Northeast and 51st Street Pumping Stations at the same time. To bypass both of these pumping stations during the TOU period and possibly over the entire day at lower demand periods, significant pumping increases at the WTP would be required in addition to making sure storage reservoirs are full before the TOU period. The increase in head at the WTP will increase energy use there but likely not enough to adversely affect the strategy of using less pumping at 51st Street and Northeast Pumping Stations by bypassing them.

Pumping through both pumping stations should be resumed immediately after the TOU period to ensure reservoirs are full by the MHD. During the TOU period, the 51st Street Reservoirs should continue to be filled by the WTP pumps so that it is full by the end of the TOU period to resume pumping at 51st Street into the Low Pressure Zone.

However, to have a storage buffer in case of drastic demand changes, it is recommended to only bypass both Northeast and 51st Street Pumping Stations when total system demand is less than 70 MGD. Having automated valves at 51st Street Pumping Station for bypassing it will allow for these operating modes to be modified remotely and over a relatively short period of time, if necessary.

Under short-term, mid-term and long-term demand conditions, several other improvements such as the connection from Northeast Reservoir to Vine Street, transmission main replacement and parallel Vine Street to "A" Street transfer main would be necessary to make bypassing both pumping stations possible.

After the transmission main between Northeast Pumping Station and Vine Street Reservoir is completed by 2025, more water can be routed along that path from the WTP High Service Pumping Station to Vine Street and "A" Street Reservoirs.

The future replacement of one of the transmission mains for the WTP with a main with a higher pressure limit will allow approximately 40 MGD to be transferred straight to "A" Street allowing bypassing 51st Street at least through the mid-term planning period.

6.2 Distribution Pumping Stations

6.2.1 West Vine Street Pump Station

The West Vine Street Pumping Station has an eddy current coupling on a pump which acts as a variable frequency drive. The eddy current coupling controls are no longer working, making it impossible to run the pump. In addition, the drive is inefficient and creates excessive heat in this mode of operation.

Since the field winding and other mechanical components of the drive and pump appear to be in fair condition, the drive should be able to be converted to a constant speed drive. This would involve either using a direct-current power supply to supply the full speed, rated amps, to the collector rings on the drive or directly coupling the pump to the motor by fully removing the eddy current coupling and using a replacement mechanical coupling. A capital improvement project was included in the immediate improvements to return this pump to service.

6.2.2 Merrill Street Pumping Station

Merrill Street Pumping Station is not currently functioning and has too little capacity to run efficiently. This pumping station is a good candidate for decommissioning and demolition.

Merrill Street Pumping Station was evaluated in the hydraulic model and was determined not necessary for future operations. Additional Belmont Service Level pumping stations in the future would be to serve a hydraulically disconnected area with the same hydraulic grade line in the short-term and the southern area of the service level in the long-range plan. However, neither of these is needed due to pumping capacity lost by decommissioning Merrill Street Pumping Station.

There is not a lot of supply redundancy across Salt Creek in the Belmont Service Level. Removing the Merrill Street Pumping Station could further reduce redundancy. A future waterline supply in the Belmont Service Level is recommended to address this supply issue to the Nebraska Innovation Campus.

Merrill Street Pumping Station is recommended for decommissioning in the immediate or short-term CIP phases, and a decommissioning/demolition project is included in the capital improvement program. Any salvageable material within the pumping station is recommended for reuse, recycling or sale. If Merrill Street is decommissioned, the surge standpipe at the site should be kept to continue to manage transient pressures in the transfer main.

6.2.3 Permanent Pumping Stations for Cheney and Northwest 12th Street

Due to the temporary nature of the existing Cheney and Northwest 12th Street Pumping Stations, permanent replacement pumping stations should be installed based on remaining life of the package pumping stations and estimated growth. Currently, there are reliability issues with the VFDs especially at Northwest 12th Street Pumping Station. Cheney Pumping Station is underground and therefore difficult to maintain. The design life was considered to be around 20 years for the package stations and they are now over 10 years through their design life. Replacement pumping stations should be above ground permanent structures and have VFDs on all pumps. These are recommended as immediate capital improvement projects.

The replacement for the Cheney Pumping Station, the Yankee Hill Pumping Station, is recommended to be located at the Yankee Hill Tank site and built in the immediate improvements phase. It is planned initially as an 12 MGD firm capacity (18 MGD installed) pump station with room for one or two additional future pumps for a future firm capacity of 16 MGD firm capacity (24 MGD installed). The existing Cheney Pumping Station could be kept online for several years after the replacement is built if additional redundancy is desired. A Cheney Pumping Station decommissioning/demolition project is included in the mid-term improvements for when it fails.

The replacement for the Northwest 12th Street Pumping Station is recommended to be located at the same location as the existing pumping station and is built into the immediate improvements phase. It is planned as an 8 MGD firm (12 MGD installed) pump station with room for one additional future pump. The existing Northwest 12th Street Pumping Station should be decommissioned immediately after the replacement is completed since they will be located at the same site and the existing VFDs seem to be failing.

6.2.4 South 56th Street Pumping Station Decommissioning

The South 56th Street Pumping Station is currently not operated and based on the modeling it does not appear to be needed. When pumps are operated to utilize more of the reservoir volume, it has proven difficult to refill the tank and results in on-going low pressure condition in the High Service Level. However, it has been suggested by LWS to remove this pumping station from service since it is not expected to be run in the future. Prior to removal/demolition, further field testing of the station should be conducted to determine its operational value. A capital improvement project has been included in the immediate improvements phase to remove the pumps and VFDs from the pumping station and to salvage them somewhere else in the system, if possible.

6.2.5 Variable Speed Pumps at Pioneers, Belmont and Southeast Pumping Stations

In the Belmont and Southeast Service Levels, pressure variations are significant when pumps start up without VFDs. Some local industries have reported issues with their fire protection systems due to these pressure variations as pumps turn on and off. Additionally, Belmont and Southeast Pumping Stations discharge into large transmission mains and as a result, there are cavitation issues. The pumps in these two stations are operating off their pump curves because of the reduced downstream head conditions. Therefore, operations at these two stations are limited to use of only the large pumps to control cavitation. The current operating procedures work around the cavitation issues but do not provide a long-term solution to be able to run the smaller pumps in the stations.

It is recommended to start with the Pioneers Pumping Station for the addition of VFDs. Although more expensive initially, VFDs are recommended instead of eddy current drives or discharge control valves due to their comparative inefficiencies. The VFDs would match pump curves to the existing and future system head curves. VFDs should be installed on all of the pumps in the pumping stations to maximum flexibility of operations and enable the smaller pumps to be used during lower flow conditions. At a minimum, VFDs should be added at Pioneers Pumping Station as those are the only ones used at this time. A capital improvement project has been included in the immediate improvements phase to implement this recommendation.

If the VFD installation is successful at Pioneers Pumping Station, VFD addition to Belmont and Southeast Pumping Stations should be evaluated and installed on the smaller pumps, if deemed cost-benefit effective, so that they can be used again during lower flow conditions without cavitation.

6.3 Distribution Pipelines

Several areas of the distribution system had excessive headloss under future MHD scenarios. Generally areas around pumping stations, namely Vine Street and “A” Street, have higher headloss due to the amount of water coming to and from these facilities. Although these high headloss are not prolonged, some improvements to help support future growth and system resiliency are recommended. Several capital improvement projects are included in the capital improvement program for distribution mains to provide redundancy and relief to high headloss areas by providing another path for water distribution.

6.3.1 Nebraska Innovation Campus

Service to the Nebraska Innovation Campus development at full build out was examined in the hydraulic model including supply to the area. The layout of future mains from the Innovation Campus Water System Study by Olsson Associates (2012) was added to the hydraulic model. According to the hydraulic model, MHDs and fire flows (3,500 gpm) are sufficiently supplied by the planned connections to the area.

In the study, Merrill Pumping Station was planned as a redundant backup to the other two supplies to the area. However, Merrill Pumping Station is being considered for decommissioning and would no longer available as a redundant source to the campus. Instead of using Merrill Pumping Station as a backup supply to the Nebraska Innovation Campus area, an immediate improvement that would include a water main from approximately Highway 6 into the Innovation Campus pipe network is recommended.

Additionally, due to its condition, corrosive soils the area, and the need for relocation, a short-term improvement that would include a water main replacement and relocation of the existing pipeline crossing Salt Creek to the Innovation Campus pipe network is recommended. This short-term water main is recommended to be completed by 2023, depending on the condition of the existing main.

6.4 Additional Pressure Monitoring

LWS maintains a pressure monitoring system throughout the distribution system. The locations of the pressure transmitters are at critical locations based upon historical data for mainly low pressure areas. There are a total of twenty existing locations where pressures are monitored within the distribution system including ten located at pumping and reservoir facilities.

6.4.1 Review of System Pressures

Distribution system improvements were evaluated on the basis of meeting current and future water requirements while maintaining a minimum pressure of 40 psi under MHD conditions. A minimum pressure of 30 psi was judged acceptable if it was caused by high ground in a small area that could not be supplied from another service level with a higher operating hydraulic gradient.

Some areas may experience low pressures that may not meet the Ten States Standards recommended minimum of 35 psi, but they exceed the Nebraska Department of Health minimum required pressure of 30 psi. For these areas, LWS should continue monitoring and install pressure monitors to determine precise conditions throughout the year, especially during high demand periods. New booster districts and expansion of areas for new development should be established with a goal of providing a minimum pressure of 40 psi under maximum hour conditions.

The review of system pressures from the maximum hour simulations indicate that pressures less than 40 psi occur along transmission mains, suction mains near pump stations, near storage tanks and a couple of smaller areas within the distribution system. Pressures less than 30 psi occur along transmission mains, on suction mains near pump stations, and near storage tanks, as expected. No major areas within the distribution system were found in the model results in the distribution system through Year 2040 with the recommended improvements. A few additional pressure monitoring stations located in lower pressure areas that develop through the planning period are recommended in the distribution system to monitor on-going performance and headloss through the system and as additional data for model validation.

There are several areas that result in high pressures (greater than 120 psi) under ADD conditions through Year 2040; all are in the High and Belmont Service Levels on the fringes of the Low Service Level.

Areas of high pressure such as these are not uncommon in many systems. These areas should be monitored and coordinated with the occurrence of main break rates. No specific recommendations are made in this report to reduce these pressures other than additional pressure monitors in the areas where no existing pressure monitor is located. However, if these areas become problematic consideration should be given in the future to methods to remediate these areas of high pressures such as PRV stations or zone boundary modifications.

6.4.2 Recommended Pressure Monitoring Locations

Additional monitoring locations are recommended to provide feedback on low and high pressures. Three low pressure and one high pressure monitoring locations are recommended in each improvement phase. The pressure monitoring locations can be built at any time as recommended in each phase; however, all four locations recommended in each phase should be constructed in one project for potential cost savings.

7.0 Recommended Improvements

Based on the findings of the steady state hydraulic analyses, the water age analyses, the fire flow analyses, operational and efficiency analyses, and the Year 2060 long-range plan, an updated CIP was prepared for each of the planning periods. This CIP includes budget costs and is staged and prioritized to identify reinvestment needs and improvements for additional capacity and reliability through Year 2060. Recommended improvements to address rehabilitation/replacement projects are prioritized and listed separately in *Chapter 6* with the exception of main upsizing for fire flow deficiencies and rehabilitation of two transmission mains.

Alignments shown for the recommended improvement mains are approximate locations. Specific locations for the mains in right-of-ways or easements should be determined during the preliminary design process. Improvement mains in undeveloped areas are subject to location change to conform to growth patterns and actual development. Factors that may accelerate or delay improvement mains include availability of right-of-way, scheduling of street improvements, and construction of other utilities.

For residential service it is recommended that the City continue its general policy of installing minimum sizes of 16-inch mains on a 1-mile grid and 12-inch mains on half-section alignments, adjusted to accommodate local street patterns. Minimum design standards recommended by LWS are 6-inch for residential, 8-inch for commercial, and 12-inch for industrial areas.

7.1 Cost Estimates

In any engineering study that develops a capital improvements program, it is necessary to make estimates of the project costs required to implement the program. To that end, basic cost data must be obtained or developed for each type of construction and system components laid out in sufficient detail to permit determination of approximate project costs.

Inherently, CIP cost estimates vary depending on the phase of the project when they are developed, which determines the level of detail and the expected accuracy of the estimate. The Association for the Advancement of Cost Engineering International (AACE International) Recommended Practices, specifically Document No. 18R-97, outlines typical cost estimate accuracies based on the overall status of the project. The cost estimates for the Transmission and Distribution Systems improvements should be considered Project Definition (Estimate Classification 5) level estimates with an expected accuracy of +100 to -50 percent.

The total project cost necessary to complete a project consists of expenditures for land acquisition, construction costs, all necessary engineering services, contingencies, and such overhead items as legal, administrative and financing services.

The cost of land acquisition is not included in the project costs presented in this report. In most cases, the construction of pipelines will not require purchase of private property or acquisition of easements. Pipeline routes, insofar as possible, follow public streets and roads. Although land or easement acquisition is a significant activity that determines whether a project occurs, the cost is generally a small portion of the overall program cost.

Construction costs cover the material, equipment, labor and services necessary to build the proposed project. Prices used in this study were obtained from a review of previous reports and pertinent sources of construction cost information. Construction costs used in this report are not intended to represent the lowest prices which may be achieved but rather are intended to represent a median of competitive prices submitted by responsible bidders.

Such factors as unexpected construction conditions, the need for unforeseen mechanical and electrical equipment, and variations in final quantities are a few examples of items that can add to planning level estimates of project cost. To cover such contingencies, an allowance of 25 percent of the construction cost has been included.

Engineering services may include preliminary investigations and reports, site and route surveys, geotechnical and foundation explorations, preparation of design drawings and specifications, engineering services during construction, construction observation, construction surveying, sampling and testing, start-up services, and preparation of operation and maintenance manuals. Overhead charges cover such items as legal fees, financing fees, and administrative costs. The costs presented in this report include a 15 percent allowance for engineering services, legal, and administrative costs.

7.2 Recommended Phased Improvements

Following development of the long-range plan, a series of analyses were conducted to develop a recommended capital improvement program with phased improvements to resolve current deficiencies, to meet projected demands, and to improve water quality. Through prioritization of hydraulic and non-hydraulic system needs as well as discussions with LWS, the recommended improvements were organized into the four phases. The phases of the program are summarized below.

- The immediate improvements (FY 2014/2015 to FY 2019/2020) are those that have been identified as higher priority as a result of their immediate need or as a result of currently anticipated development. Immediate improvements also include improvements to correct identified fire flow deficiencies. Immediate improvements are given a CIP identification of IM- for immediate and a sequential number based on their recommended timing.
- Improvements recommended to meet Year 2025 demand conditions are referred to as short-term improvements (FY 2020/2021 to FY 2025/2026). The short-term improvements will extend service to the limits of the Tier I – Priority B area. Short-term improvements are given a CIP identification of ST- for short-term and a sequential number based on their recommended timing.
- Improvements recommended to meet Year 2040 demand conditions are referred to as mid-term improvements (FY 2026/2027 to FY 2040/2041). The mid-term improvements will extend service to the limits of the Tier I – Priority C area. Mid-term improvements are given a CIP identification of MT- for mid-term and a sequential number based on their recommended timing.
- Improvements recommended to meet Year 2060 demand conditions and provide service beyond the Tier I limits out to the Tier II limits are referred to as long-term improvements (FY 2041/2042 to FY 2060/2061). Long-term improvements are given a CIP identification of LT- for long-term and a sequential number based on their recommended timing.

7.2.1 Immediate and Short-term Improvements

Immediate and short-term recommended improvements will provide service to the limits of Tier I – Priority A and B development areas.

The immediate improvements should be viewed as a subset of the short-term improvements. They are recommended to correct existing deficiencies, and provide a partial list of projects that should be included in the next 6 years of the LWS CIP. Some short-term improvements that are not specifically identified as immediate will also be included in the 6-year CIP and should be prioritized based on known or anticipated development.

7.2.1.1 Immediate Improvements (FY 2014/2015 to FY 2019/2020)

The LWS CIP from the 2007 Master Plan was reviewed and compared to completed projects since 2007. Results from the base year analyses and fire flow analyses as part of this Facilities Plan which identified existing deficiencies were reviewed and addressed. From this review, immediate improvements were identified and prioritized. The recommended immediate capital improvements are summarized in Table 7-1.

The immediate improvements that should be included in the 6-year immediate CIP and include the following:

- **Yankee Hill Road Main Improvements (IM-1):** Required to connect pathway of water along Yankee Hill Road, reduce headloss in Southeast Service Level, and provide additional distribution capacity for future growth. Benefits include higher pressures, reduced velocities and headloss, and increased redundancy.
- **West Vine Street Pump Modifications to Remove Eddy Current Coupling (IM-2):** Required to return pump to working order as a constant speed pump due to failed Eddy Current Drive. Benefits include returning pump to service and increased flexibility of operations.
- **Valve Replacement and Automation at 51st Street Reservoirs and Pumping Station (IM-4):** Required due to condition of existing valves and desire to automate valves to bypass the 51st Street Pumping Station with approximately 14-15 MGD from the WTP straight to the Low Service Level. Benefits include increased operational flexibility, temporary shutdown of at least one 51st Street Reservoir, energy savings, and water age improvements.
- **Nebraska Innovation Campus Redundant Supply Immediate Improvement (IM-5):** Required to provide an additional supply to the forthcoming redevelopment of the area since Merrill Street Pumping Station is planned for decommissioning. Benefits include a redundant supply to two vulnerable creek crossings with no additional creek crossing.
- **Merrill Street Pumping Station Decommissioning/Demolition (IM-7):** Required due to small pumping station which is no longer used. The surge standpipe on the Merrill Street property will be kept in service for the transfer main. Benefits include less maintenance, reduced operational complexity and freed up resources.
- **Northeast Pump Modifications to Remove Eddy Current Coupling (IM-8):** Required due to eddy current drive generating excessive heat and related inefficiencies and to convert pump to constant speed. Benefits include increased pumping capacity without excessive heat and increased pump efficiency.
- **South 56th Street Pumping Station Decommissioning (IM-9):** Required to take pumping station out of service by removing pumps and VFDs (which should be salvaged, if possible). Benefits include reduced maintenance efforts and reuse of the building as a potential maintenance storage facility.

- **Southeast Pumping Station PRV Vault to High Service Level (IM-10):** Required for redundancy of bleed back of water from the Southeast Service Level to High Service Level. Benefits include increased operational flexibility and increased ability to take the South 56th or Southeast Reservoirs offline for maintenance.
- **Control Valve or Similar Water Quality Improvement at Pioneers Pump Station (IM-11):** Required for increasing turnover in the Pioneers Reservoir. Benefits included reduced water age, less stagnant water and increased ability to control level in Pioneers Reservoir.
- **Valve Vault Relocation to "A" Street Reservoirs Site (IM-15):** Required due to existing valve condition and difficult access for maintenance. Benefits include increased transfer control, enhanced operations, and better access to the vault.
- **12.0 MGD Firm (18.0 MGD Installed) Permanent Yankee Hill Pumping Station (IM-16):** Required due to deteriorating condition of Cheney Pumping Station that is underground and not easily accessible for maintenance. This pumping station should include rechlorination facilities to boost chlorine residuals in this area of the system. Benefits include increased pumping capacity to meet future demands and improved operations and maintenance ability of Cheney Service Level pumping facilities.
- **Cheney Pumping Station Decommissioning/Demolition (IM-19):** Required due to deteriorating condition of existing Cheney Pumping Station. Benefits include reduced maintenance and addition of permanent pumping facilities for the Cheney Service Level.
- **3.0 MGD Firm (6.0 MGD Installed) Booster Pumping Station at I-80 (IM-20):** Required due to eventual development of area north of I-80. Timing of this project can float some depending on when first development begins in this area. Benefits include expanded service area to support new development, additional customers, and use of existing pipeline crossing under I-80.
- **Improvement Mains for Development north of I-80 (IM-21):** Required due to eventual development of area north of I-80 and delivery of water to and from the new Booster Pumping Station feeding this area. Timing of this project can float some depending on when first development begins in this area. Benefits include expanded service area to support new development and additional customers.
- **8.0 MGD Firm (12.0 MGD Installed) Permanent Northwest 12th Street Pumping Station (IM-24):** Required due to deteriorating condition of existing Northwest 12th Street Pumping Station. This pumping station should include rechlorination facilities to boost chlorine residuals in this area of the system. Benefits include increased pumping capacity to meet future demands and improved operations and maintenance ability of Northwest Service Level pumping facilities.
- **Northwest 12th Street Pumping Station (Fallbrook) Decommissioning/Demolition (IM-25):** Required due to deteriorating condition of existing Northwest 12th Street

Pumping Station and its scheduled replacement. Benefits include reduced maintenance and addition of permanent pumping facilities for the Northwest Service Level.

- **Replace 10.1 MGD Pump with 20.2 MGD Pump at East Vine Street Pumping Station to Southeast Service Level (IM-26):** Required due to increasing demands in the Southeast Service Level. A VFD should be included with this pump replacement. Benefits include increased pumping capacity from Vine Street Pumping Station to Southeast Service Level.
- **Pioneers Pumping Station VFD Additions (IM-27):** Required to smooth out operations in the Belmont Service Level. Benefits include improved flow control, reduced cavitation issues, and controlled pressure variations during pump start-up and shut-down.
- **Tank Mixing Study and Improvements (IM-31):** Required due to high water age and stagnant water within storage reservoirs within the distribution system. Benefits include in-depth understanding of flow within tanks, mixing needs, and improvements to water age and quality.
- **Immediate Distribution System Extensions (IM-32):** Required due to normal growth of the system and service area from development surrounding the City. Benefits include expanded service area to support new development and additional customers.
- **Immediate Pressure Monitoring Stations (IM-33):** Required due to monitoring needs of low and high pressure areas in the distribution system. Benefits are increased awareness of system performance, improved operations warning system, and additional data for hydraulic model calibration.
- **Immediate Automatic Flushing Hydrants for Chlorine Residual (IM-34):** Required due to dead end areas of system with lower chlorine residual. Flushing hydrants should include chlorine residual sampling units to only flush what is needed to increase residual back to desired level. Benefits are increased water quality and reduced water age in the distribution system.
- **Immediate Fire Flow Improvements (IM-3, 6, 12, 13, 14, 15, 16, 17, 18, 22, 23, 28, 29, and 30):** Recommended improvements are identified to address potential fire flow deficiencies. Actual fire flow goals should be verified and additional hydraulic modeling should be completed for all potentially deficient areas before implementing any improvements.

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Table 7-1 Recommended Immediate Transmission and Distribution Systems Capital Improvements

Year	CIP ID	Description	Type of Improvement	Current Cost Basis[1]	Future Cost Basis (3%)[2]	Future Cost Basis (5%)[3]
Pumping, Storage, and Transmission						
2014	IM-2	West Vine Street Pump Modifications to Remove Eddy Current Coupling	Pumping	$72,000	$75,000	$76,000
2016	IM-4	Valve Replacement and Automation at 51st Street Reservoirs and PS	Valving	$324,000	$355,000	$376,000
2020	IM-7	Merrill Street Pumping Station Decommissioning/Demolition	Pumping	$216,000	$266,000	$304,000
2016	IM-8	Northeast Pump Modifications to Remove Eddy Current Coupling	Pumping	$72,000	$79,000	$84,000
2016	IM-9	South 56th Street Pumping Station Decommissioning	Pumping	$75,000	$82,000	$87,000
2015	IM-15	Valve Vault Relocation to "A" Street Reservoirs Site	Valving	$259,000	$275,000	$286,000
2017	IM-16	12.0 MGD Firm (18.0 MGD Installed) Permanent Yankee Hill Pumping Station	Pumping	$4,313,000	$4,855,000	$5,243,000
2018	IM-19	Cheney Pumping Station Decommissioning/Demolition	Pumping	$216,000	$251,000	$276,000
2018	IM-20	3.0 MGD Firm (6.0 MGD Installed) Booster Pumping Station at I-80	Pumping	$971,000	$1,126,000	$1,240,000
2019	IM-24	8.0 MGD Firm (12.0 MGD Installed) Permanent Northwest 12th Street Pumping Station	Pumping	$2,875,000	$3,433,000	$3,853,000
2020	IM-25	Northwest 12th Street Pumping Station (Fallbrook PS) Decommissioning/Demolition	Pumping	$216,000	$266,000	$304,000
2019	IM-26	Replace 10.1 MGD Pump with 20.2 MGD Pump at East Vine Street PS to Southeast SL	Pumping	$1,829,000	$2,184,000	$2,452,000
2019	IM-27	Pioneers Pumping Station VFD Additions	Pumping	$173,000	$207,000	$232,000
2019	IM-31	Tank Mixing Study and Improvements	Quality	$575,000	$687,000	$771,000

Year	CIP ID	Description	Type of Improvement	Current Cost Basis[1]	Future Cost Basis (3%)[2]	Future Cost Basis (5%)[3]
Distribution						
2014	IM-1	Yankee Hill Road Main Improvements	Distribution	$4,430,000	$4,563,000	$4,652,000
2015	IM-5	Nebraska Innovation Campus Redundant Supply Immediate Improvement	Distribution	$860,000	$913,000	$949,000
2016	IM-10	Southeast Pumping Station PRV Vault to High SL	Valving	$144,000	$158,000	$167,000
2016	IM-11	Control Valve or Similar Water Quality Improvement at Pioneers Pump Station	Quality	$259,000	$284,000	$300,000
2018	IM-21	Improvement Mains for Development north of I-80	Distribution	$1,946,000	$2,256,000	$2,484,000
2014-2019	Various	Immediate Fire Flow Improvements	Fire Flow	$2,076,000	$2,342,000	$2,533,000
2014-2019	IM-32	Immediate Distribution System Extensions	Distribution	$8,537,000	$9,482,000	$10,164,000
2014-2019	IM-33	Immediate Pressure Monitoring Stations	Monitoring	$138,000	$156,000	$167,000
2014-2019	IM-34	Immediate Automatic Flushing Hydrants for Chlorine Residual	Quality	$69,000	$79,000	$85,000
-	-	**Total Immediate Projects**	-	**$30,645,000**	**$34,374,000**	**$37,085,000**
Average Cost Per Year					**$5,729,000**	**$6,181,000**

Notes:

1. *Engineering and Contingency estimates are included in each item at a value of Contingency 30% and Engineering 20% of the item cost.*
2. *Inflated to projected year dollars at 3% per year inflation rate.*
3. *Inflated to projected year dollars at 5% per year inflation rate.*

7.2.1.2 12-year Short-term Improvements (FY 2020/2021 to FY 2025/2026)

The recommended short-term capital improvements are summarized in Table 7-2. The short-term improvements should be included in the 12-year short-term CIP and include the following:

- **Parallel Transmission Main from Northeast Pumping Station to Vine Street Reservoirs (ST-1):** Required due to increasing demand in the system and to completely connect the high service pumping to Vine Street. Benefits include increased operational flexibility and avoid additional re-pumping at Northeast Pumping Station.
- **Cheney to Southeast PRV Station for Water Quality (ST-2):** Required to boost chlorine residuals in the southern portion of the Southeast Service Level adjacent to the Cheney Service Level from Yankee Hill Pumping Station rechlorination. Benefits include increased circulation between service levels alleviating dead ends, reducing water age, and boosting chlorine residuals.
- **Water Main on Northwest 56th Street (ST-3):** Required for redundancy and looping and to support future growth to northwest area in the Belmont Service Level. Benefits include increase system resiliency and support of future development.
- **Belmont to Low PRV Station (ST-4):** Required due to fire flow deficiencies at the edge of the Low Service Level in this vicinity. Benefits included additional supply during high flow and fire flow periods and reduced estimated fire flow deficiencies.
- **Northwest Reservoir (2 MG Elevated) and Pipeline for Northwest Service Level (ST-5):** Required due to lack of redundancy to Northwest 12th Street Pumping Station and need for floating storage in the Northwest Service Level. Benefits include smoother operation of Northwest 12th Street Pumping Station, service level supply redundancy, emergency storage for multiple service levels, and more uniform service level pressures.
- **Add 20.9 MGD WTP High Service Pump (ST-6):** Required due to growing system demands and need to deliver more water to Vine Street Reservoir. Benefits include increased operational flexibility and high service pumping capacity into the transmission system.
- **Nebraska Innovation Campus Redundant Supply Short-term Improvement (ST-7):** Required to provide replace and relocate the 1963 water supply to the forthcoming redevelopment of the area since it is vulnerable due to condition and corrosive soils. Benefits include replacing an aging that crosses a creek and relocating around several planned facilities.
- **Adams Street Reservoir (5 MG above-grade) and Pipeline for High Service Level (ST-8):** Required to support growing demands in High Service Level. Benefits include increased storage to support development and operational flexibility from Vine Street and “A” Street Pumping Stations into the High Service Level.

- **Short-term Distribution System Extensions (ST-9):** Required due to normal growth of the system and service area from development surrounding the City. Benefits include expanded service area to support new development and additional customers.
- **Short-term Pressure Monitoring Stations (ST-10):** Required due to monitoring needs of low and high pressure areas in the distribution system. Benefits are increased awareness of system performance, improved operations warning system, and additional data for hydraulic model calibration.
- **Short-term Automatic Flushing Hydrants for Chlorine Residual (ST-11):** Required due to dead end areas of system with lower chlorine residual. Benefits are increased water quality and reduced water age in the distribution system.

Table 7-2 Recommended Short-term Transmission and Distribution Systems Capital Improvements

Year	CIP ID	Description	Type of Improvement	Current Cost Basis[1]	Future Cost Basis (3%)[2]	Future Cost Basis (5%)[3]
Pumping, Storage, and Transmission						
2020-2022	ST-1	Parallel Transmission Main from Northeast PS to Vine St Reservoir	Transmission	$24,840,000	$31,477,000	$36,730,000
2022	ST-5	Northwest Reservoir and Pipeline for Northwest SL (2 MG elevated)	Storage	$6,799,000	$8,872,000	$10,548,000
2023	ST-6	Add 20.9 MGD WTP High Service Pump	Transmission	$1,503,000	$2,020,000	$2,449,000
2024	ST-8	Adams Street Reservoir and Pipeline for High SL (5 MG above-grade)	Storage	$13,285,000	$18,390,000	$22,722,000
Distribution						
2020	ST-2	Cheney to Southeast PRV Station for Water Quality	Quality	$144,000	$239,000	$274,000
2021	ST-3	Water Main on NW 56th Street	Distribution	$1,246,000	$1,579,000	$1,841,000
2021	ST-4	Belmont to Low PRV Station	Valving	$144,000	$183,000	$213,000
2023	ST-7	Nebraska Innovation Campus Redundant Supply Short-term Improvement	Distribution	$1,127,000	$1,515,000	$1,836,000
2020-2025	ST-9	Short-term Distribution System Extensions	Distribution	$22,574,000	$29,934,000	$36,011,000
2020-2025	ST-10	Short-term Pressure Monitoring Stations	Monitoring	$138,000	$186,000	$223,000
2020-2025	ST-11	Short-term Automatic Flushing Hydrants for Chlorine Residual	Quality	$69,000	$95,000	$112,000
-	-	**Total Short-term Projects**	-	**$71,869,000**	**$94,490,000**	**$112,968,000**
Average Cost Per Year				**$15,748,000**	**$18,828,000**	

Notes:
1. *Engineering and Contingency estimates are included in each item at a value of Contingency 30% and Engineering 20% of the item cost.*
2. *Inflated to projected year dollars at 3% per year inflation rate.*
3. *Inflated to projected year dollars at 5% per year inflation rate.*

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7.2.2 25-year Mid-term Improvements (FY 2026/2027 to FY 2040/2041)

The recommended mid-term capital improvements are summarized in Table 7-3. The mid-term improvements should be included in the 25-year mid-term CIP and include the following:

- **Transfer Pipeline from Vine Street to “A” Street (MT-1)**: Required due to delivering additional water to “A” Street from Vine Street by gravity from Northeast Reservoir to Vine Street. Benefits include redundant transfer main from Vine Street to “A” Street, increased supply to west side of system, and increased operational flexibility to draw water away from 51st Street Pumping Station and Reservoir.
- **Add 20.2 MGD Pump at East Vine Street Pumping Station to Southeast Service Level (MT-2):** Required due to expected growth in Southeast Service Level. Benefits include increased firm pumping capacity at East Vine Street Pumping Station to Southeast Service Level and improved operational flexibility.
- **Add 5.0 MGD Pump at Pioneers Pumping Station (MT-3):** Required due to expected growth in Belmont Service Level. Benefits include increased firm pumping capacity at Pioneers Pumping Station to Belmont Service Level and improved operational flexibility.
- **Transmission Main Replacement from Platte River WTP to “A” Street (MT-4):** Required due to estimated end of life of the Transmission Main. Benefits include new main with higher pressure limit and capacity (approximately 40 MGD) to continue transfer of water from 51st Street to “A” Street.
- **40.0 MGD Firm (60.0 MGD Installed) Pump Station at Northeast Reservoir and Pumping Station (MT-5):** Required to pump additional future supply from the Platte River Water Treatment Facility from the transmission mains and Northeast Reservoir into the High Service Level. Benefits include pumping directly to the High Service Level from Northeast Reservoir, improved operational flexibility, ability to meet growing demand, and increased water delivery from the Platte River Water Treatment Facility into the distribution system to allow delay of the future Missouri River supply.
- **Belmont Connector Main (MT-6):** Required to connect Belmont Service Level areas together and to eventually decommission the I-80 and North 56th Pumping Station. Benefits include redundant supply to service level areas, access to floating storage, and eventual removal of a pumping station.
- **Replace Pump at WTP with 20.9 MGD Pump (MT-7):** Required due to projected demands and increase in head. Benefits include increased pumping capacity at higher head in high service pumping station at the WTP.
- **Add 6.0 MGD Pump in Yankee Hill Pumping Station for 18.0 MGD Total Firm and 24.0 MGD Total Installed (MT-8):** Required due to expected growth in Cheney Service Level. Benefits include increased firm pumping capacity at Yankee Hill Pumping Station to Cheney Service Level and improved operational flexibility.

- **Cheney II Reservoir (3 MG elevated) and Pipeline for Cheney Service Level (MT-9)**: Required due to projected demand growth in Cheney Service Level. Benefits include additional storage in Cheney Service Level to support development and increase reservoir maintenance flexibility.
- **Mid-term Distribution System Extensions (MT-10):** Required due to normal growth of the system and service area from development surrounding the City. Benefits include expanded service area to support new development and additional customers.
- **Mid-term Pressure Monitoring Stations (MT-11):** Required due to monitoring needs of low and high pressure areas in the distribution system. Benefits are increased awareness of system performance, improved operations warning system, and additional data for hydraulic model calibration.
- **Mid-term Automatic Flushing Hydrants for Chlorine Residual (MT-12):** Required due to dead end areas of system with lower chlorine residual. Benefits are increased water quality and reduced water age in the distribution system.

Table 7-3 Recommended Mid-term Transmission and Distribution Systems Capital Improvements

Year	CIP ID	Description	Type of Improvement	Current Cost Basis[1]	Future Cost Basis (3%)[2]	Future Cost Basis (5%)[3]
Pumping, Storage, and Transmission						
2027	MT-1	Transfer Pipeline from Vine St to “A” St	Transmission	$16,256,000	$24,589,000	$32,186,000
2028	MT-2	Add 20.2 MGD Pump at East Vine Street Pumping Station to Southeast SL	Pumping	$1,743,000	$2,716,000	$3,624,000
2029	MT-3	Add 5.0 MGD Pump at Pioneers Pumping Station	Pumping	$360,000	$578,000	$786,000
2031	MT-4	Transmission Main Replacement from Platte River WTP to “A” Street	Transmission	$64,032,000	$109,011,000	$154,101,000
2033	MT-5	40.0 MGD Firm (60.0 MGD Installed) Pump Station at Northeast Reservoir and Pumping Station	Transmission	$12,938,000	$23,368,000	$34,329,000
2037	MT-7	Replace Pump at WTP with 20.9 MGD Pump	Pumping	$1,503,000	$3,056,000	$4,848,000
2039	MT-8	Add 6.0 MGD Pump (18.0 MGD Total Firm/24.0 MGD Total Installed) in Yankee Hill Pumping Station	Pumping	$432,000	$932,000	$1,537,000
2040	MT-9	Cheney II Reservoir and Pipeline for Cheney SL	Storage	$8,777,000	$19,497,000	$32,769,000
Distribution						
2035	MT-6	Belmont Connector Main	Distribution	$3,696,000	$7,082,000	$10,812,000
2033	MT-10	Mid-term Distribution System Extensions	Distribution	$27,667,000	$49,970,000	$73,409,000
2033	MT-11	Mid-term Pressure Monitoring Stations	Monitoring	$138,000	$250,000	$367,000
2033	MT-12	Mid-term Automatic Flushing Hydrants for Chlorine Residual	Quality	$69,000	$125,000	$184,000
-	-	**Total Mid-term Projects**	-	**$137,611,000**	**$241,174,000**	**$348,952,000**
Average Cost Per Year					**$16,078,000**	**$23,263,000**

Notes:
1. *Engineering and Contingency estimates are included in each item at a value of Contingency 30% and Engineering 20% of the item cost.*
2. *Inflated to projected year dollars at 3% per year inflation rate.*
3. *Inflated to projected year dollars at 5% per year inflation rate.*

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7.2.3 50-year Long-term Improvements (FY 2041/2042 to FY 2060/2061)

The recommended long-term capital improvements are summarized in Table 7-4. The long-term improvements should be included in the 50-year long-term CIP and include the following:

- **Add 20.2 MGD Pump at West Vine Street Pumping Station to High Service Level (LT-1):** Required to meet projected demands of High Service Level. This pump would fill last open slot at this pumping station. Benefits include increased firm capacity of West Vine Street Pumping Station to High Service Level to support development and improved operational flexibility.
- **Transmission Main Rehabilitation or Replacement (LT-2):** Required due to estimated end of life of the Transmission Main. Benefits include extended service life of main to continue transfer of water from the WTP and 51st Street Reservoir.
- **East Supply Transmission Main to Vine Street Reservoir (LT-3):** Required due to delivering the future water supply from east of City, as described in *Chapter 3 - Water Supply*, to the Vine Street Reservoir. Benefits include additional source of water directly into Low Service Level to support development and increased operational and system supply flexibility.
- **Booster Pumping Station at I-80 Street Decommissioning/ Demolition (LT-4):** Required due to expected deteriorating condition of existing I-80 Pumping Station. Benefits include reduced overall system maintenance and ability to add South Belmont Pumping Station to support growth in southwest area of system.
- **Southwest Reservoir (4 MG above-grade) and Pipeline for Belmont Service Level (LT-5):** Potentially required due to projected storage requirements in the Belmont Service Level, if projected growth occurs. Benefits include increased storage in Belmont Service Level and support of growth in the south areas of the service level.
- **Saltillo Road Reservoir (3 MG above-grade) and Pipeline for High Service Level (LT-6):** Potentially required due to projected storage requirements in the High Service Level, if projected growth occurs. Benefits include increased storage in High Service Level and support of growth in the south areas of the service level.
- **Adams Street Reservoir II (5 MG above-grade) and Pipeline for High Service Level (LT-7):** Potentially required due to projected storage requirements in the High Service Level, if projected growth occurs. Benefits include increased storage to support development and operational flexibility from Vine Street and “A” Street Pumping Stations into the High Service Level.
- **5.0 MGD Firm (8.0 MGD Installed) South Belmont Pumping Station to Belmont Service Level (LT-8):** Required due to expected growth in south area of Belmont Service Level. Benefits include increased firm capacity to deliver water to the Belmont

Service Level to support development and to spread out supply points across the service level.

- **Rokeby Reservoir (5 MG above-grade) and Pipeline for Southeast Service Level (LT-9):** Potentially required due to projected storage requirements in the Southeast Service Level, if projected growth occurs. Benefits include increased storage in Southeast Service Level and support of growth in the south and east areas of the service level.
- **Northwest Reservoir II (3 MG elevated) and Pipeline for Northwest Service Level (LT-10):** Potentially required due to projected storage requirements in the Northwest Service Level, if projected growth occurs. Benefits include increased storage in Northwest Service Level and support of growth in the north and west areas of the service level.
- **Long-term Distribution System Extensions (LT-11):** Required due to normal growth of the system and service area from development surrounding the City. Benefits include expanded service area to support new development and additional customers.

Table 7-4 Recommended Long-term Transmission and Distribution Systems Capital Improvements

Year	CIP ID	Description	Type of Improvement	Current Cost Basis[1]	Future Cost Basis (3%)[2]	Future Cost Basis (5%)[3]
Pumping, Storage, and Transmission						
2041	LT-1	Add 20.2 MGD Pump at West Vine Street Pumping Station to High SL	Pumping	$1,452,000	$3,323,000	$5,693,000
2042	LT-2	Transmission Main Rehabilitation/Replacement	Transmission	$59,693,000	$140,671,000	$245,705,000
2044	LT-3	East Supply Transmission Main to Vine Street Reservoir	Transmission	$2,816,000	$7,041,000	$12,780,000
2045	LT-4	Booster Pumping Station at I-80 Decommissioning/Demolition	Pumping	$216,000	$557,000	$1,030,000
2046	LT-5	Southwest Reservoir and Pipeline for Belmont SL (4 MG above-grade)	Storage	$4,506,000	$11,952,000	$22,545,000
2048	LT-6	Saltillo Road Reservoir and Pipeline for High SL (3 MG above-grade)	Storage	$5,167,000	$14,540,000	$28,502,000
2050	LT-7	Adams Street Reservoir II and Pipeline for High SL (5 MG above-grade)	Storage	$5,529,000	$16,506,000	$33,625,000
2052	LT-8	5.0 MGD Firm (8.0 MGD Installed) South Belmont Pumping Station to Belmont SL	Pumping	$1,618,000	$5,125,000	$10,849,000
2054	LT-9	Rokeby Reservoir and Pipeline for Southeast SL (5 MG above-grade)	Storage	$7,075,000	$23,772,000	$52,299,000
2058	LT-10	Northwest Reservoir II and Pipeline for Northwest SL (3 MG elevated)	Storage	$4,284,000	$16,201,000	$38,492,000
Distribution						
2050	LT-11	Long-term Distribution System Extensions	Distribution	$70,710,000	$211,086,000	$430,017,000
-	-	**Total Long-term Projects**	-	**$163,066,000**	**$450,774,000**	**$881,537,000**
Average Cost Per Year					**$22,539,000**	**$44,077,000**

Notes:
1. *Engineering and Contingency estimates are included in each item at a value of Contingency 30% and Engineering 20% of the item cost.*
2. *Inflated to projected year dollars at 3% per year inflation rate.*
3. *Inflated to projected year dollars at 5% per year inflation rate.*

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Lincoln Water System Facilities Master Plan

Chapter 6 - Water Main Replacement Program

May 2014

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Abbreviations and Acronyms

AWWA	American Water Works Association
AWWARF	AWWA Research Foundation (now the WaterRF)
Cdf	Cumulative Density Function
CI	Cast Iron Pipe
CIP	Capital Improvements Program
CIPP	Cured in Place Pipe
CML	Cement Mortar Lining
CMMS	Computerized Maintenance Management System
City	City of Lincoln
DIP	Ductile Iron Pipe
ft	Feet
GIS	Geographic Information System
HDR	HDR Engineering, Inc.
LWS	Lincoln Water System
2007 Master Plan	2007 Facilities Master Plan
Master Plan	2013 Facilities Master Plan
NRCS	Natural Resources Conservation Service
PDF	Probability Density Function
PE	Polyethylene
PVC	Polyvinyl Chloride Pipe
WaterRF	The Water Research Foundation (formerly AWWARF)

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1.0 Introduction

Chapter 6 – Water Main Replacement Program provides a systematic approach to establishing criteria for annual main replacements. This chapter reviews the current main replacement program, evaluates the projected service life of the mains based several factors, and makes recommendations for program improvement moving forward.

2.0 Current Water Main Replacement Program

2.1 System Overview

The Lincoln Water System (LWS) distribution system consists of a wide range of pipe sizes, ages and materials. As of the end of fiscal year 2012, LWS has approximately 1,200 miles of water mains ranging in size from 4 to 60inches in diameter. The oldest mains in the system were installed in the late 1800's.

Current design standards allow service connections on mains up to and including 16-inches in diameter. Table 2-1 shows the length of main in the system by pipe diameter. Small diameter water distribution mains (16-inch diameter and smaller), comprise approximately 89 percent of all water mains.

Table 2-1 Water Main Data by Pipe Diameter

Pipe Diameter	Sum of Miles in Active System (miles)	Percentage of Total
<6"	69	6%
6"	603	51%
8-10"	105	9%
12"	161	14%
14-16"	116	10%
18-24"	59	5%
>24"	72	6%
Grand Total	**1,185**	**100%**

Note:
The results in the Table 2-1 include the mileage of the active system only. Table 2-1 includes lengths of Water Distribution Network (from GIS) that was provided by LWS (does not include lengths of GIS Production Network). LWS's Main Break Spreadsheet reports 1256 miles in the system.

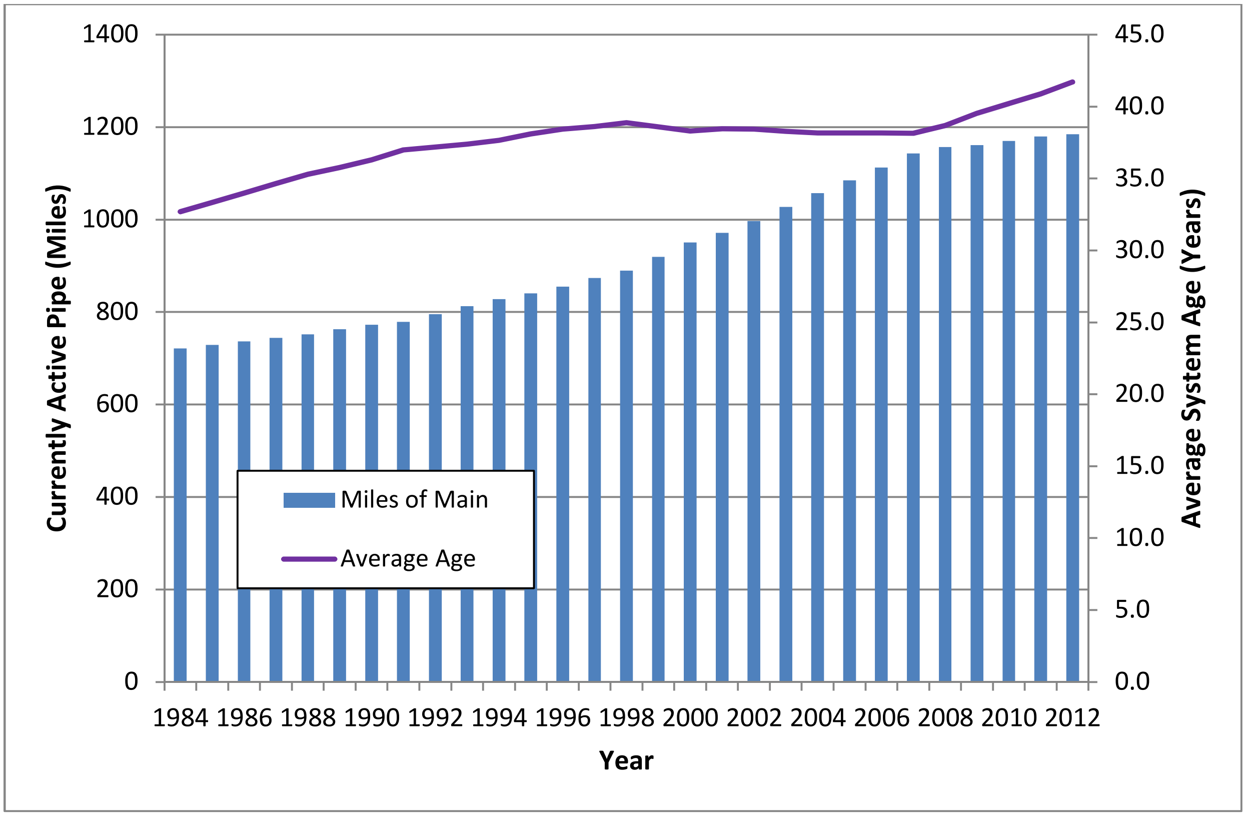
For the period 1984 through 2012, LWS added approximately 460 miles to their distribution system, or approximately 16 miles per year in the 29-year period. Table 2-2 displays the total length of water mains in the LWS system, length of system that has been abandoned, length of small diameter mains, and average age of the system for each given year.

Table 2-2 Annual Distribution System Data

Year	Total Length of All Mains in Active System (miles)	Length of Small Diameter Mains, 16-inch and below (miles)	Average Age (years)
1984	722	640	32.7
1985	729	645	33.3
1986	737	653	34.0
1987	744	659	34.6
1988	752	667	35.3
1989	763	678	35.8
1990	773	688	36.3
1991	779	694	37.0
1992	795	711	37.2
1993	813	726	37.4
1994	828	742	37.7
1995	841	752	38.1
1996	855	766	38.4
1997	873	783	38.6
1998	890	797	38.9
1999	919	826	38.6
2000	951	855	38.3
2001	971	873	38.5
2002	997	897	38.4
2003	1,027	922	38.3
2004	1,057	947	38.2
2005	1,085	971	38.2
2006	1,113	996	38.2
2007	1,143	1,016	38.1
2008	1,157	1,028	38.7
2009	1,161	1,031	39.5
2010	1,170	1,040	40.2
2011	1,180	1,049	40.9
2012	1,185	1,054	41.7

Note:
The results in the Table 2-2 include the mileage of the active system only. Table 2-2 includes lengths of Water Distribution Network (from GIS) that was provided by LWS (does not include lengths of GIS Production Network). LWS's Main Break Spreadsheet reports 1256 miles in the system.

Figure 2-1 displays the average age of water mains in the LWS distribution system from 1984 to 2012 and the associated number of miles in the active system.



Note:
The results in the Figure 2-1 include the mileage of the active system only. Figure 2-1 includes lengths of Water Distribution Network (from GIS) that was provided by LWS (does not include lengths of GIS Production Network). LWS's Main Break Spreadsheet reports 1256 miles in the system.

Figure 2-1 Average Age of LWS Water Main Distribution System

As of the end of 2012, the average age of water mains in the system was approximately 42 years. As shown in Figure 2-1, the average age of the water main in system is gradually increasing with time which is typical for most utility systems.

As a result, a water main replacement program is required to maintain a level of service for LWS customers. LWS implemented a systematic water main replacement program in 1992. Since that time, LWS has worked to refine this program with prioritization and focused replacement of the water mains that pose the greatest risk and consequence of failure in the system. Chapter 6 of the 2013 Facilities Master Plan (Master Plan) describes and evaluates the current water main replacement program and provides further analysis of the system that can be used to refine prioritization factors and means of replacement.

2.2 Main Replacement Project Selection

One of the greatest challenges faced by any water utility is how to best spend the available funds to maintain a safe and reliable distribution system. This is becoming more challenging as many of the various pipe materials installed over the years will near the end of their useful service lives. To manage water main assets and maintain an acceptable level of service, LWS has instituted a water main replacement program which includes prioritizing the water main replacement needs through an asset ranking structure to identify the highest priority replacement projects.

To facilitate this process, LWS uses an asset ranking form. This form is used to prioritize potential projects based on several criteria including:

- Level of Service Consequence
- Damage Consequence
- Water Main Condition and Failure Risk

This form results in an asset rank score. LWS uses this score along with other data including number of breaks per 1,000 ft, capacity, fire flow improvements, etc., to prioritize projects for replacement. Opportunity projects are also considered based on planned roadway projects. When a street reconstruction or widening project is planned, the water main in the area is evaluated for replacement depending on water main age and break history. Coordination of roadway projects and water main replacement projects limits disruptions to the community and provides cost savings.

The use of this process has prioritized the water main replacement needs for LWS and established a budget of $4.0 million in fiscal year 2013. This equates to 5 miles (on average) of water main replacement projects ranging in size from 4-inch to 16-inch, or about 0.4 percent of the overall distribution system. This replacement rate is similar to the rates of many other water utilities. At the current replacement rate, the asset inventory is renewed, on average, every 240 years. Table 2-3 displays the current budget for the period 2013 through 2017, and the corresponding estimated length of replacement.

Table 2-3 Water Main Data by Pipe Diameter

Year	CIP Budget[1] ($ million)	Approximate Length of Replacement[2] (miles)
2013	4.0	4.6
2014	4.3	4.8
2015	4.6	4.8
2016	5.1	5.1
2017	5.6	5.4

Notes:

1. CIP Budget numbers shown are based on LWS's 2012 – 2018 CIP.

2. The estimated lengths are based on the assumption that the replacement projects will cost approximately the same (on a linear foot basis) as the replacement projects that occurred in 2013 (plus inflation).

2.3 System Component Inspection Procedures

2.3.1 Valves

LWS has 26,203 active valves in the system including hydrant branch valves. These valves are in place to isolate sections of the system in the event of a main break or system maintenance. A valve is considered to be inoperable if any of the following conditions exist:

- The valve cannot be located quickly in the field.
- The valve cannot be accessed or operated due to obstructions or material in the valve box.
- The valve cannot be fully closed to isolate the water main.

If a valve is determined to be inoperable, the cause is noted and a repair order is created. To ensure proper operation, valves sized 6-inch through 24-inch are inspected on a 4-year cycle. Inspection procedures include the following:

- Locate the valve.
- Attempt to access the operating nut.
    - If the operating nut is accessible, then the valve is exercised. While exercising valves (weather permitting), a hydrant should be flowing in a location that will create flow through the valve for debris removal and discharge from the system.
    - If the operating nut is not accessible, then the box is cleaned either during the inspection or by work order if cleaning equipment is not available during the inspection.
- Valves that are not located or have an inaccessible operating nut are recorded as inoperable.
- Hydrant valves are located and documented, but are not exercised since they do not affect customer service or fire protection provided.

2.3.2 Hydrants

There are 11,001 hydrants through the LWS distribution system. Due to public safety, all fire hydrants are inspected on an annual basis. The following is an overview of the inspection procedures:

- Remove weeds, vegetation, and other obstructions from the hydrant.
- Determine if the hydrant needs to raised or lowered.
- Inspect the breakaway device for damage.

- Operate the hydrant at a low flow rate until the water runs clear.
- Check the hydrant for drain down.
- Clean the threads on the nozzle caps and lubricate the threads on the large steamer cap and one small cap.
- Check the outlet-nozzle-cap chains and cable for free action, adjust if necessary to ensure free action.
- Make all necessary field repairs.
- If field repair cannot be easily made, ensure the repair request is documented on the hydrant inspection check list.
- Tag any inoperable hydrants and report immediately to supervisor and support staff to initiate a repair order.
- Notify Fire and Rescue of any hydrants that are out of service.

Due to the configuration of some of the existing hydrants in the system, pressure testing can damage the hydrants. As a result, LWS no longer pressure tests hydrants, but does conduct routine inspections which are recorded in the LWS computerized maintenance management system (CMMS).

2.3.3 Corrosion Protection

Cathodic protection has been installed on several larger diameter (16-inch, 24-inch and larger) iron pipelines in recent years. LWS staff reports that the earliest pipe to be cathodically protected is the 54-inch raw water main from the island to the 48-inch interconnection. The 54-inch transmission main from the Platte River Water Treatment Facility to Greenwood is also cathodically protected. The cathodic protection is inspected on these two mains annually by LWS. The 60-inch transmission main installed in 2009 from Greenwood to the Northeast Pumping Station is also cathodically protected.

The distribution system contains a number of cathodically protected mains. The goal for testing of the cathodic protection in the distribution system is to complete the testing every 5 years. However, over time many of the test stations have been damaged or cannot be located. LWS is currently working on restoration of the test station locations and will begin the testing cycle when the restoration is complete. LWS has also standardized the design and installation of the test stations in recent years to facilitate inspection of the cathodic protection. Prior to the standardization, the design of the test stations varied from project to project.

2.3.4 Flushing

LWS does not currently have a systematic flushing program for the water system. Temporary hydrant flushers are utilized in the Airpark area to reduce water age and maintain chlorine residuals in that portion of the system. If customer complaints regarding water quality are received, LWS responds to these complaints to mitigate any issues. Based on discussions with LWS staff, the water quality complaints have been minimal and as long as water quality objectives are being met, a systematic flushing program will not be implemented.

2.4 Current Level of Implementation

One of the goals of this Master Plan is to review the recommendations made in the 2007 Facilities Master Plan (2007 Master Plan) regarding the LWS water main replacement program and assess their level of implementation. Based on the system needs and resources available, it is common that all recommendations are not fully implemented and it is up to LWS staff to determine which of the recommendations would have the greatest benefit for the system. The critical factor is that program is constantly being refined to address the needs of the system and maintain a level of service for customers.

Recommendations from the 2007 Mater Plan and the current status are shown in Table 2-4.

Table 2-4 Current Level of Implementation

Recommendations from the 2007 Master Plan	Current Status
Address/evaluate funding needs moving into the future.	Addressing funding needs for the future is an ongoing process for LWS. LWS has made significant progress in recent years in increasing the level of investment associated with water main replacement and infrastructure renewal. The 2007 Master Plan recommended a funding level based on benchmarking of $6.92 million annually. From 2011-2013, LWS spent $4 million per year (on average) on water main replacement projects and was able to maintain the level of service in the system.
Establish formal replacement criteria for small diameter rehabilitation or replacement.	From section 2.2 of this Chapter, criteria exist for prioritizing the replacement projects. A rating system is used to rank the pipes. From this ranking, candidate lists of projects are developed and further refined to reach a final list of replacement projects.
Implement a large diameter inspection program.	LWS has not started a large diameter inspection program. Recommendations as to how LWS may implement such a program are provided in this Chapter.
Modify valve inspection schedule to inspect large diameter (16-inch and larger) valves annually.	16-inch to 24-inch valves are inspected on a 4-year cycle (see Section 2.3.1). Valves greater than 24-inch are not routinely exercised.
Modify the hydrant inspection procedures and record keeping conforming to American Water Works Association (AWWA) Manual	The hydrant inspection procedures provided by LWS conform to AWWA Manual M17.

Recommendations from the 2007 Master Plan	Current Status
M17.	
Implement a flushing program for dead-end mains and entire system per the 1995 AWWA Research Foundation (AWWARF) report, *Implementation and Optimization of Distribution Flushing Programs.*	LWS reports no systematic flushing program as water quality issues have not been experienced system wide. LWS has installed automatic flushers in areas known to have water age issues or low chlorine residuals. Per the 2003 AWWARF Report, “Investigation of Pipe Cleaning Methods,” system-wide flushing is not necessary if water quality objectives are met.
Implement leak detection and monitoring program.	LWS reports no leak detection and monitoring program.
Implement a corrosion protection monitoring program.	LWS has procedures for monitoring corrosion protection (See Section 2.3.3)

This comparison shows that LWS has made significant progress in their water main replacement and maintenance program. The 2007 Master Plan focused primarily on benchmarking to evaluate the LWS water main replacement program. Since that time, LWS has spent significant time populating and maintaining a Geographic Information System (GIS) database with information that can be used to more fully evaluate the LWS water main replacement program and offer recommendations for refining project prioritization. The following sections discuss this analysis.

3.0 Pipe Asset Evaluation

The primary purpose of the pipe asset evaluation conducted for the Master Plan was to progress towards using main break data to prioritize rehabilitation and replacement of pipelines. Results of the analysis include recognizing and characterizing trends from past main breaks in the system.

Statistical analysis of main breaks combined with pipe asset information can be a valuable tool for planning the rehabilitation and replacement program of a water distribution system. Condition assessment based on breaks correlated to known pipe attributes can be used where little, if any, direct inspections have been performed. Direct inspections of water distribution pipelines are not common because they tend to be invasive, disruptive, and costly. Therefore, by comparing pipe break information of similar pipes under similar conditions, the entire system can be examined for areas of potential pipe failure.

3.1 Summary of Existing Data Provided by LWS

The following data were provided by LWS to perform the pipe asset evaluation or main break analysis:

- Water Main Replacement Project spreadsheet which summarizes the past water main replacement projects and the projected costs of potential future projects.
- Water main break history as recorded in the CMMS
- GIS Database which includes the pipe network, break history, associated appurtenances, and other relevant features.

During review of the break database, some erroneous/duplicate breaks were identified. LWS staff performed a data clean up exercise and provided a second break database in GIS.

In consultation with LWS staff, it was determined that the data maintained in their CMMS reflected the most accurate system wide record of break data. As a result this data was used for system wide summaries of the break data. It was further agreed with LWS staff that the GIS database would be used for the statistical analysis and modeling conducted as a part of the assessment in order to obtain the spatial distribution of breaks. It is recognized that their GIS database, while improved with the City of Lincoln's (City) effort to clean up the data, still includes some duplication of data but it was not significant enough to impact the analysis conducted for the purpose of this Master Plan.

Additionally, it was determined that the main break records from 1991 to current are relatively complete. However, break records from 1984 to 1990 reflect only a portion of breaks that occurred. Therefore, for analyses that require relatively complete data sets, break records were filtered to include the time period of 1991 through the end of 2012. Data sets for 2013 were not included (only a partial years worth of data was available at the time of this analysis).

3.2 General Assumptions and Limitations

3.2.1 Data Collection and General Data Assumptions

Recognizing the need for accurate data to assess and prioritize the water main replacement needs throughout the system, LWS has made significant progress in populating their GIS database and assessing main breaks as these occur. However, as many utility systems have experienced, the age and complexity of these systems can pose some challenges in obtaining all the data needed to complete a water main break analysis. Table 3-1 presents some of these challenges and how they were addressed for the water main break analysis completed as a part of this Master Plan. Typically (and for this study), a break is defined as an identified breach in a pipe that has been or will be repaired. Other commonly used terms in the industry that this term encompasses are "leaks", "ruptures", "failures", "repairs", and "blowouts". These terms exclude breaches at service laterals, hydrants, and valves.

Table 3-1 Water Main Data in GIS by Pipe Diameter

Challenge	Description	Project Team Strategy
Missing Main Breaks	Discrepancy between data sources in total break count for period between 1962 and 1990.	Limited data set to 1991-2012. During this period, reliable break data were available.
Identifying Pertinent Break Data	Not all breaks are associated with aging infrastructure. An example is “dig-ins” where a main break is caused by a construction crew inadvertently damaging a pipe. It is important to remove such data from the analysis as a “dig-in” is not a true assessment of the likelihood of a pipe break.	Contractor “dig-ins” were accounted for by looking at the Contractor field. If this field was “Yes”, it indicated a contractor dig-in and this break was excluded from the analysis.
Absence of Condition Data	To increase the value of failure analysis based on main break data, main break history should be supplemented by pipe attribute data. However, pipe data including size, material and age may be incomplete or inaccurate which can impair the outcome of condition assessment.	The data analyzed included the break and pipe network database. Many fields such as lifecycles (i.e., active, abandoned, proposed, or removed), material, and diameter were complete or nearly complete. Other fields such as install date and year of abandonment were not fully populated.
Associating a Break to the Pipe that Broke	The breaks are not tied to a specific pipe which could allow for errors in joining the break to the correct pipe.	HDR joined the breaks to the nearest pipe, except where the distance between the break location and the pipe was greater than 40 ft. Breaks on pipes with young ages were reviewed manually and adjusted accordingly. Breaks on pipes with negative ages were tied to the nearest abandoned or removed pipe.
Inconsistent Material Identified between Break Data and Pipe Data	After the spatial join, some inconsistencies were identified between material of pipe data and break data. This could be an indication of an error in the spatial join for that particular break.	The inconsistencies in material were minor in number. The inconsistencies were reviewed manually to see if the break should have been tied to a former pipe that was abandoned or removed.

Each time a main break occurs in the City’s water distribution system, information on the break is collected and logged into the breaks geodatabase by LWS staff. Collection of main break history started in 1962 and has continued since then. Prior to 1991, the records were taken by hand.

HDR Engineering, Inc. was given 3,441 main breaks for analysis. Table 3-2 presents a summary of the main break data that was considered as a part of the detailed spatial break analysis and modeling. For the system wide break analysis, the data recorded in Hansen reflect fewer breaks.

Table 3-2 Main Break Data in GIS

Description	Number of Breaks	Comments
Original Break data provided by LWS	3441	Main break data provided on 9/26/2013.
Breaks were not spatially joined to a pipe	(-117)	This was either due to a distance between pipe and break that was greater than 40 ft, negative break age (meaning the original pipe has already been replaced), or other reasons
Breaks occurring in 2013	(-70)	Analysis included breaks for full years only. Period of analysis was 1991-2012.
Breaks caused by Contractor “dig-ins”.	(-24)	Breaks were not due to pipe deterioration.
Breaks prior to 1/1/1991	(-215)	Inconsistent break data prior to 1991.
Total Number of Breaks Analyzed	**3015**	**Includes breaks on active, abandoned, and removed pipes.**

The locations of these breaks are shown in the Figure 3-1.

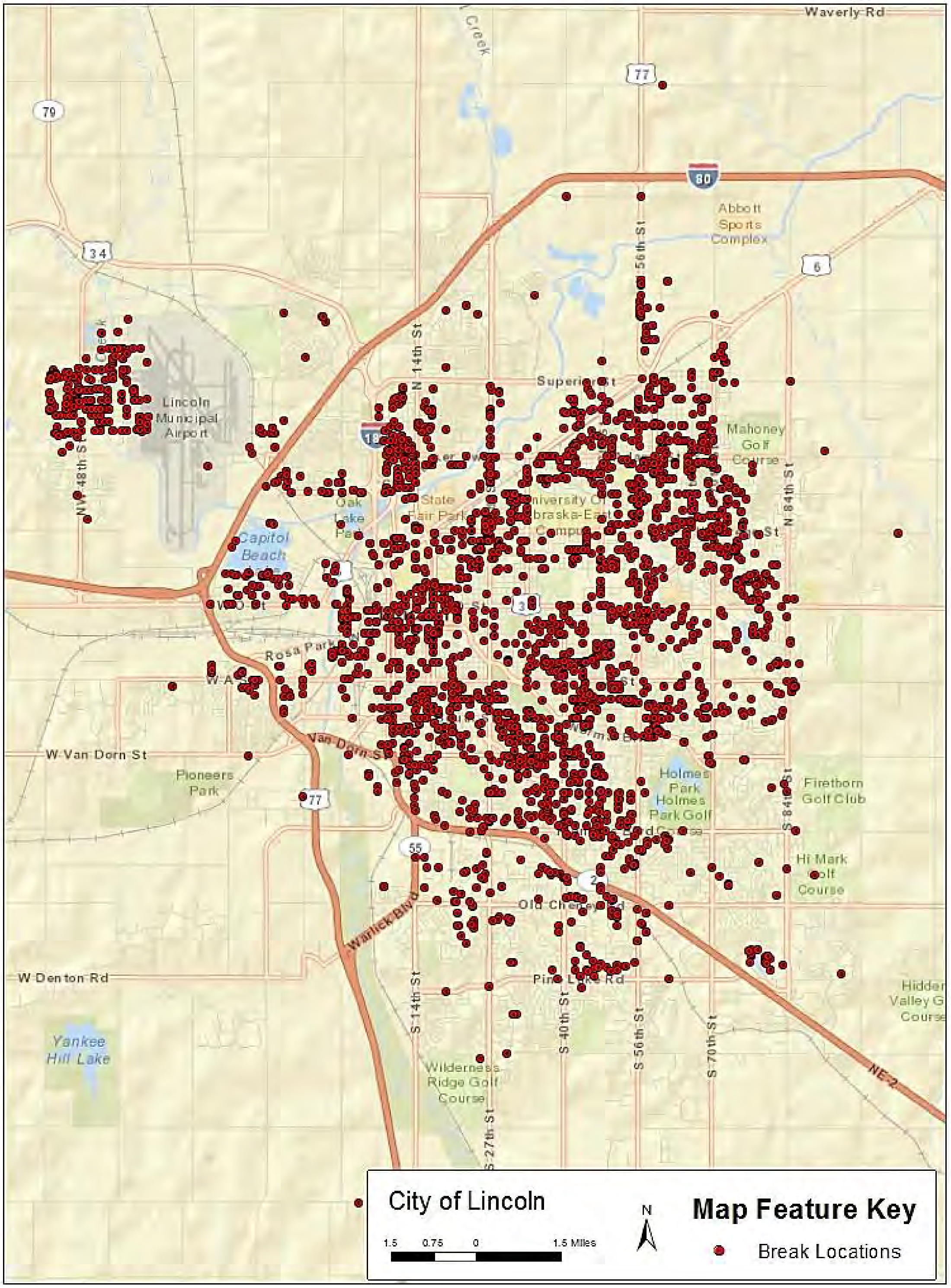


Figure 3-1 Break Location Map

This data was analyzed to determine the relative “density” of breaks which allow a visual inspection to determine areas that have higher than normal water main breaks. Figure 3-2 shows the break density in the City with a quarter-mile radius. Areas in red have a high break density as compared to the green areas with lower break densities.

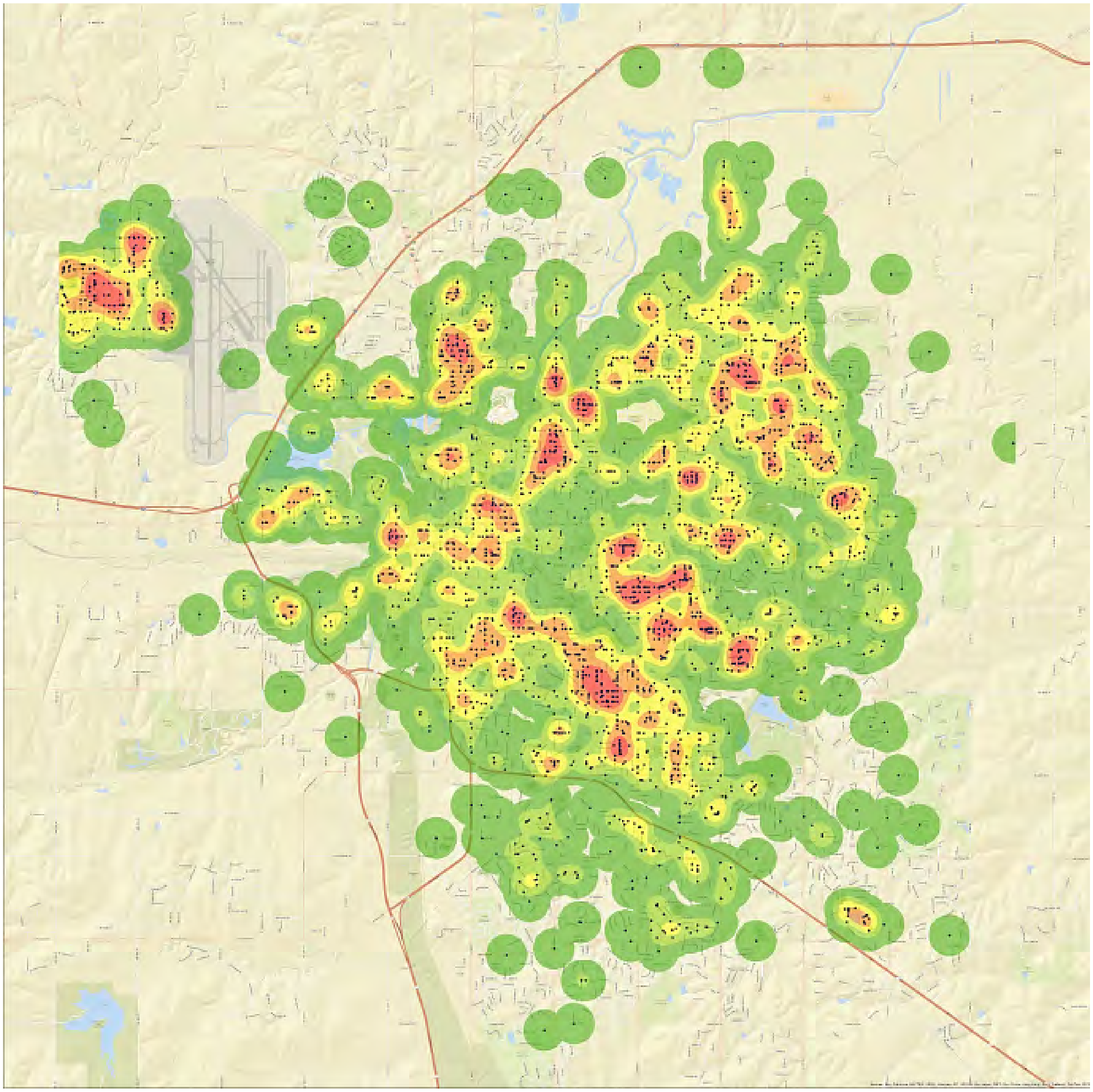


Figure 3-2 Main Break Density

3.2.2 Break and Water Main Association

To associate these break locations with a particular water main, HDR used a spatial analysis. ESRI's geo-processing tools spatially locate the pipe closest to each break point. In performing this analysis, the following steps were performed:

Pipes with a status of "proposed" were excluded in the analysis.

The "Near" and "Spatial Join" functions identified the closest water main and assigned the pipe's unique ID to the break. In addition, the spatial analysis provided the distance from the break to the nearest pipe.

Breaks located more than 40 ft from the nearest pipe were removed because the data were considered unreliable.

HDR also calculated the age of the pipe at the time of the break. If the calculation resulted in a negative age, it was assumed that this water main had previously been replaced and the break was not associated with the new water main. In these instances, the break was joined to the nearest abandoned or removed pipe. If the distance between the break and the nearest abandoned or removed pipe was more than 40 ft, the break was removed from the analysis.

HDR reviewed breaks where the pipe age was between 0 and 20 years old. These breaks were manually associated with the pipe that most likely broke.

3.2.3 Isolated Pipe Definition

Currently, LWS's pipe network (like many water utilities) is divided into distinct assets at appurtenances (valves, tees, crosses, reducers, bends, etc.) and/or pipe characteristic changes (install year, material, diameter, etc.). While this is ideal for some applications, it is not ideal for pipeline replacement planning purposes because this method dilutes the count of breaks per pipe. For proactive pipeline planning purposes, it is more appropriate to identify pipes by the smallest unit that can be hydraulically isolated and will be proactively replaced as a single project. This can be achieved systematically by identifying groups of pipes that can be hydraulically isolated through valves, dead ends, and service connections. For the purposes of this Chapter, this grouping of pipes was called the "Isolated Pipe".

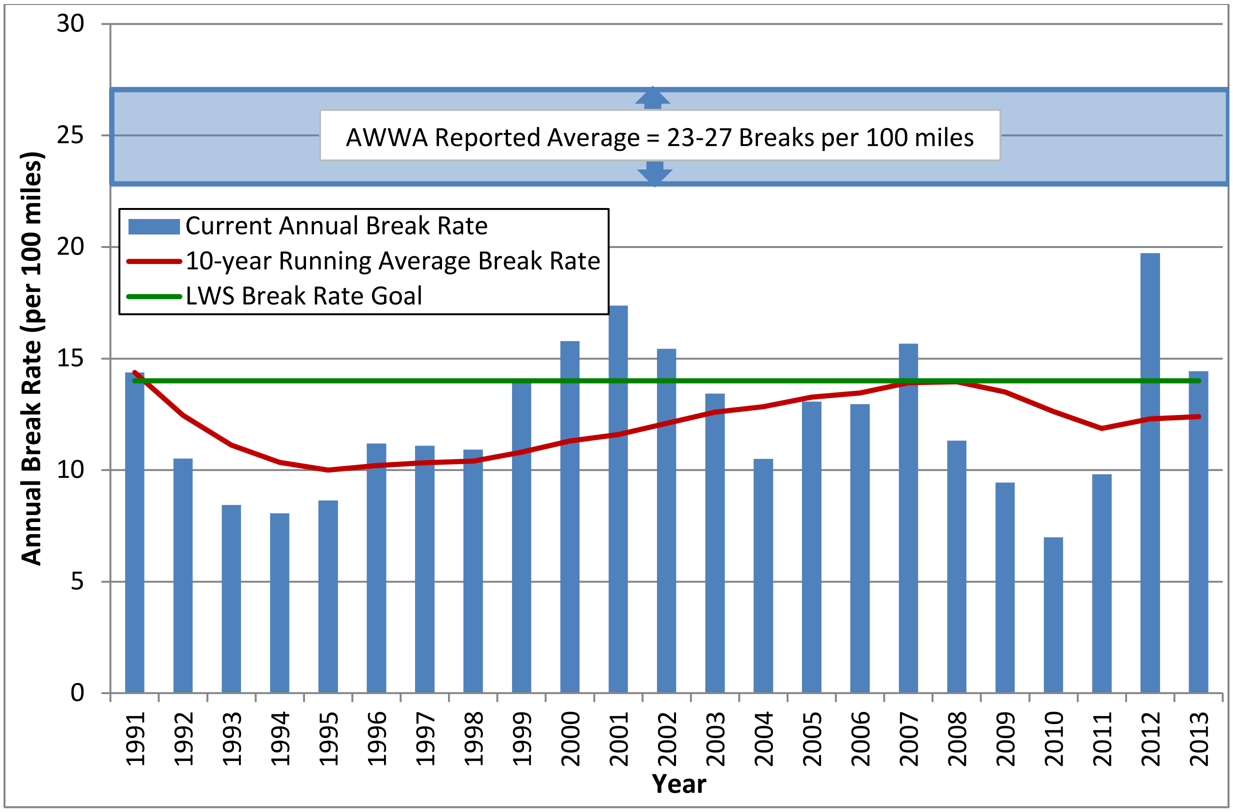
3.2.4 System Performance

Pipeline system integrity plays a pivotal role in the size of a utility's pipeline replacement program. Water main break rates are a common measure of pipeline system integrity and define a key part of pipeline system performance.

The Break Rate is calculated as the annual number of breaks divided by the system mileage, multiplied by 100. Based on an Environmental Protection Agency (EPA) commissioned study prepared by AWWA summarizing available research entitled "Distribution System Inventory, Integrity and Water Quality", the average break rate in the United States was estimated at between 23 and 27 annual breaks per 100 miles. LWS set a goal of no more than 14 breaks per 100 miles. After this goal was set, a 1995 Water Research Foundation report, “Distribution System Performance Evaluation” suggested a “reasonable goal for main breaks for a system in North America is 25 to 30 per 100 miles per year”, whereas a more recent study, “Criteria for Optimized Distribution Systems” (WaterRF Project 4109) suggested a goal of 15 or fewer breaks per 100 miles per year, to minimize the cost, disruption, and various risks associated with main failures, including the risk of pathogen entry. A lower break rate indicates better overall condition.

System performance trends are important because they indicate whether the need for break repairs is increasing. Utilities with higher break rates typically require a more robust pipeline replacement program than utilities with lower break rates to meet level-of-service goals, and other key stakeholder expectations. In addition, allowing a higher system wide break rate can have significant impacts on the staffing required to address these water main breaks.

As presented in Figure 3-3, LWS is currently performing better than the national average as reported by AWWA. Based on discussions with LWS, the current system wide break rate is the level that can be managed with their current staffing level. In the early 2000’s, the break rate was slightly above LWS’s goal of no more than 14 breaks per 100 miles. The cause of the higher break rates during this period is unknown, but break rates do tend to fluctuate significantly between seasons and between years. When severe conditions hit, it forces the weak links to expose themselves as breaks. In general, after the weak links have broken, the break rate tends to improve the following years after the severe conditions.



Notes:
The 10-year running average is calculated by averaging LWS's break rate over the past 10 years.
LWS has stated that their goal is no more than 14 breaks per 100 miles of pipe in the system.
Break rate is based on break data from the Hansen CMMS database.

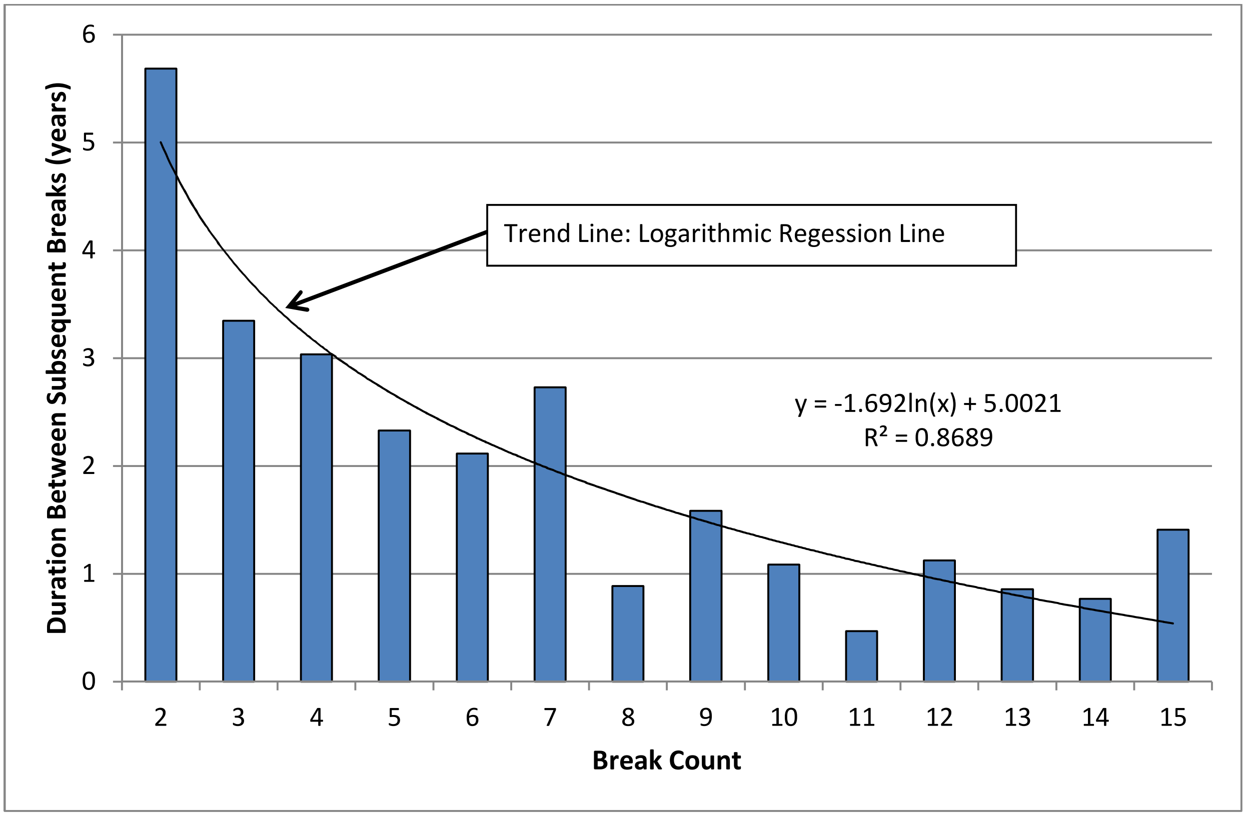
Figure 3-3 System Performance Relative to LWS Goal & WaterRF Recommended Goal

3.3 Performance of Current Water Main Replacement Program

3.3.1 Performance by Break Count

The purpose of this analysis is to determine if there is a relationship between the number of historic breaks on a pipe (i.e., Break Count) and the duration to the next break on that pipe. If this relationship is proven for a particular system, the Break Count can be a powerful indicator of future pipe performance and ultimately serve as the foundation for an effective distribution system renewal prioritization process.

Figure 3-4 shows the average duration between subsequent breaks for Isolated Pipes in the LWS system. For example, once the first break occurs, on average it takes approximately 5.7 years for the second break to occur. Once the second break occurs, on average it takes approximately 3.3 years for the third break to occur. There is a strong trend that can be drawn to show the relationship between Break Count and the duration to the next break within the LWS system (R-squared value is 0.8689). Therefore, Break Count should continue to be used in the distribution system renewal prioritization process. Additionally this relationship will be used in future sections to estimate historic break savings (projected) due to the LWS's existing water main replacement program.



Note:
Break count based on break data from GIS database.

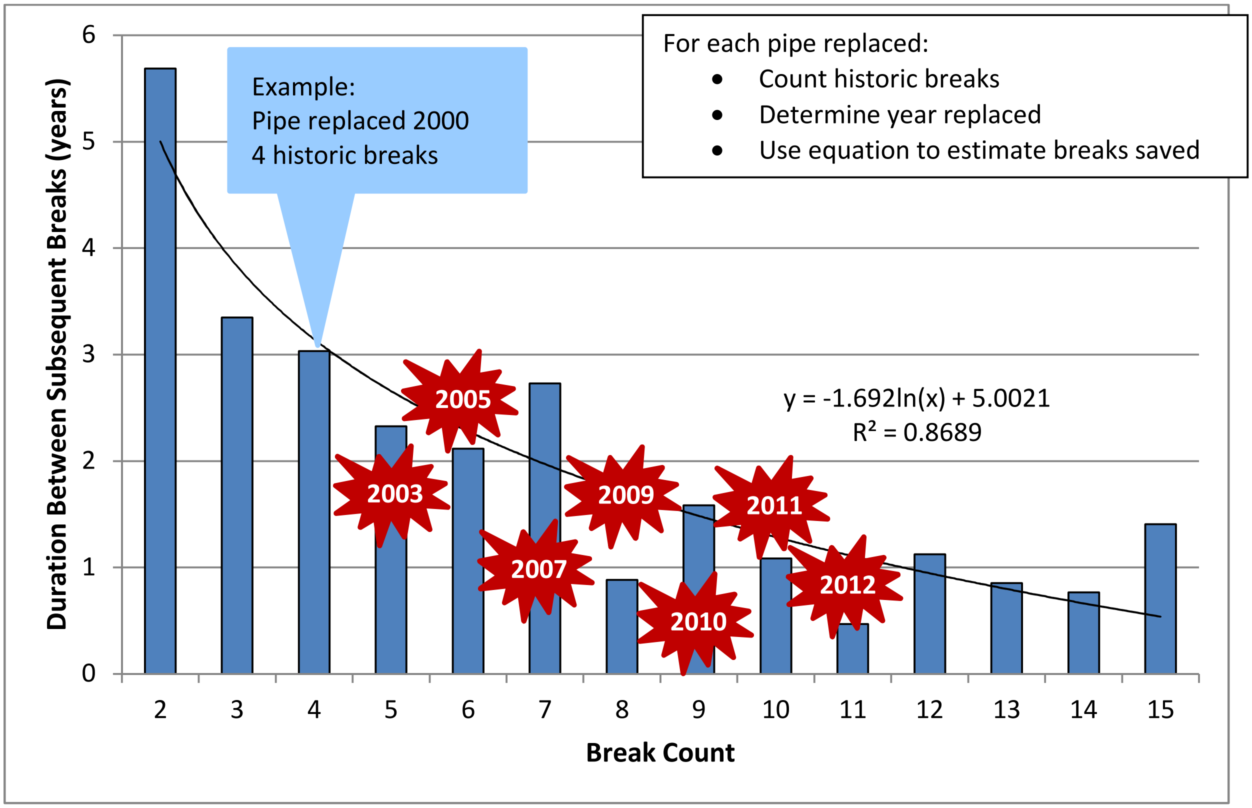
Figure 3-4 Average Pipe Performance by Break Count

Since this analysis does not calculate an annual break rate, all readily available break data from 1984 to current were used in this analysis. Break data were not readily available prior to 1984.

3.3.2 Projected Breaks Saved by Existing Main Replacement Program

The purpose of this analysis is to quantify the benefits realized from LWS's proactive water main replacement program between 1991 and 2012. The equation shown in Figure 3-4 was used to predict when subsequent breaks would have occurred on Isolated Pipes that were replaced by LWS between 1991 and 2012. For example, if an Isolated Pipe was replaced in

2000 and had four historic breaks at the time of replacement, the equation would predict avoided breaks in 2003, 2005, 2007, 2009, 2010, 2011, and 2012. This example is shown in Figure 3-5.



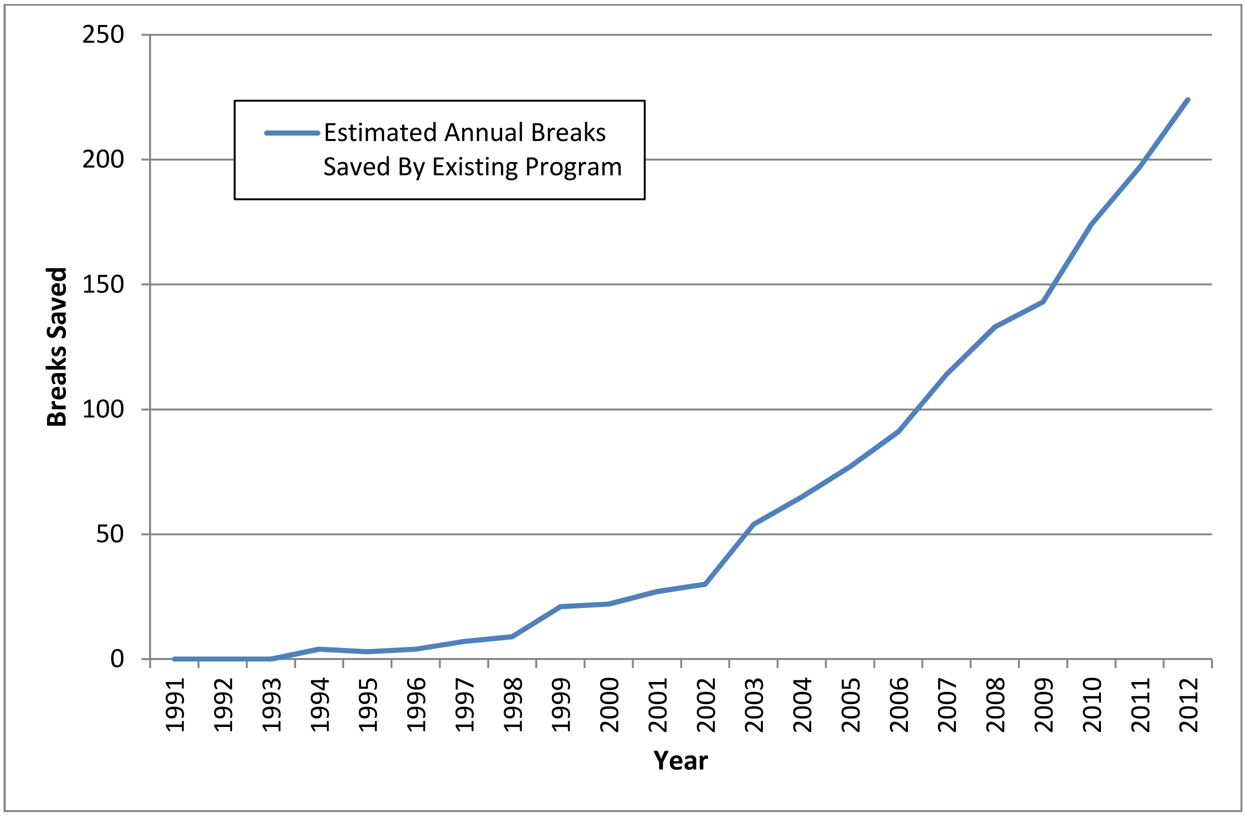
Note:
Break count based on break data from GIS database.

Figure 3-5 Average Pipe Performance by Break Count Example

At the time of this Master Plan, the pipe-specific replacement dates for projects between 1991 and 2012 were not available. For the purposes of this Master Plan, any pipe in a status of Abandoned or Removed was assumed to be abandoned or replaced in the year of the last recorded break. Based on this assumption, approximately 43 miles of pipe were replaced between 1991 and 2012.

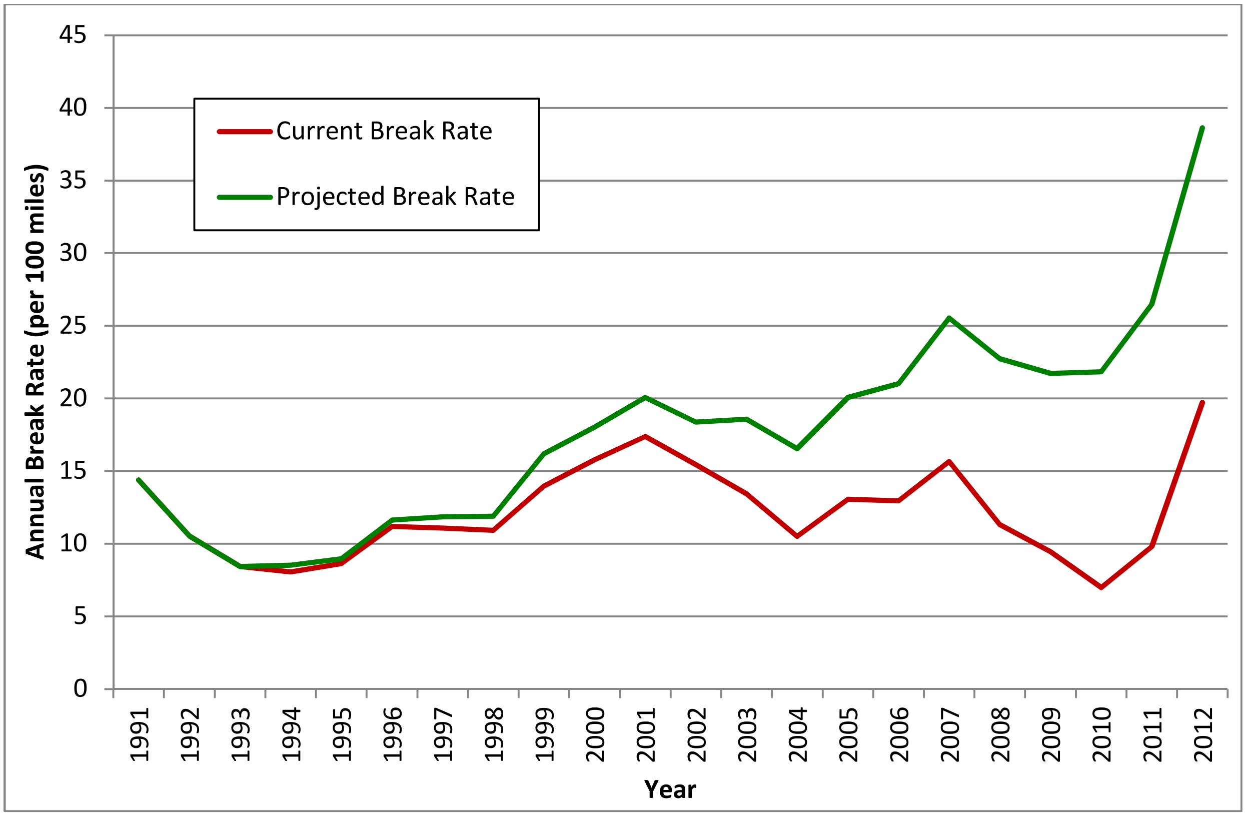
Based on this method, Figure 3-6 displays the total number of breaks avoided annually due to LWS's proactive water main replacement program between 1991 and 2012. Figure 3-7 displays how this program has impacted projected break rates over this time period. Figure 3-8 displays the cumulative total of breaks saved by the existing water main replacement program. In total, the program has saved approximately 1,400 breaks (estimated) over the 1991-2012 period.

This method predicts that annual breaks would be approximately twice as high today if LWS had not implemented their proactive replacement program between 1991 and 2012. These additional breaks would have required substantial increases in reactive response and repair crews, additional damage to adjacent roads and structures, and reduced the level of service provided to customers.



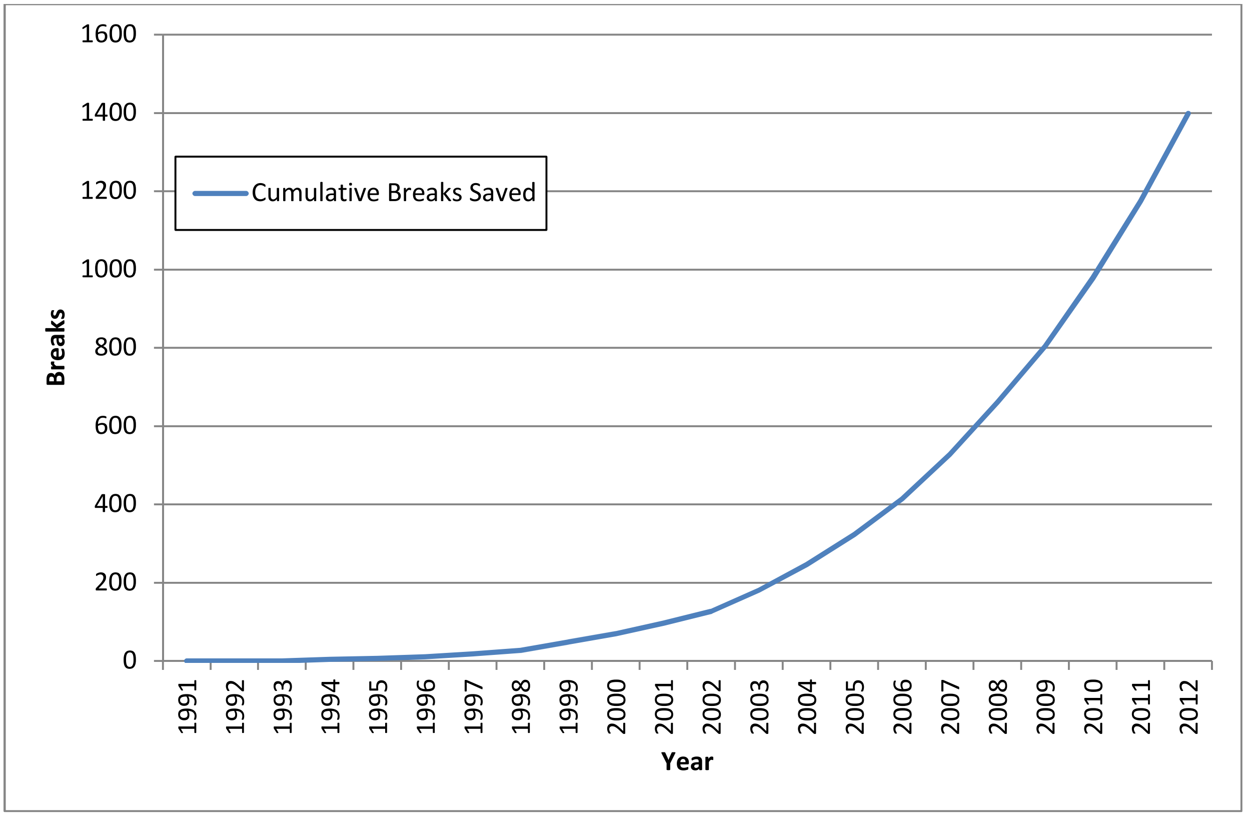
Notes:
The main breaks were capped at a maximum of 4 breaks per year per Isolated Pipe for the purpose of this analysis.
Break data obtained from Hansen CMMS database.

Figure 3-6 Estimated Annual Breaks Saved By Existing Replacement Program



Notes:
The main breaks were capped at a maximum of 4 breaks per year per Isolated Pipe for the purpose of this analysis.
Break data obtained from Hansen CMMS database.

Figure 3-7 Projected Break Rate Without Main Replacements



Notes:
The main breaks were capped at a maximum of 4 breaks per year per Isolated Pipe for the purpose of this analysis.
Bread data obtained from Hansen CMMS database

Figure 3-8 Estimated Cumulative Breaks Saved By Existing Replacement Program

As these figures show, the benefits of a proactive replacement program accumulate over time as more and more breaks are avoided.

3.4 Deterioration by Asset Class

In general, as water mains age and deteriorate, they break more often and negatively impact overall system performance. The deterioration rate of individual pipes varies significantly based on many factors. The purpose of this analysis is to leverage readily available data to determine which factors impact deterioration in the LWS system and quantify the relative strength of each factor. While it is not anticipated that we will fully understand all of the drivers for deterioration, the intent is to advance our understanding of primary drivers of deterioration in the LWS system. This information will be used to identify the useful life of the system, size a sustainable renewal budget, and prioritize condition assessment and renewal resources.

For this study, the following deteriorations factors were studied:

- Quality of Material – Inclusive of pipe material, manufacturing technique, manufacturing standards, and typical corrosion protection measured by the installation date in GIS.
- Pipe Diameter – As recorded in GIS.
- Operating Pressure – As estimated by a GIS calculation between hydraulic grade line (by service level) and elevation of each individual pipe.
- Ground Slope – As estimated by the slope of the pipe in GIS.
- Soil Corrosion Potential – As estimated by a spatial join of soil data from Natural Resources Conservation Services (NRCS) Web Soil survey and Lincoln's GIS pipe data.
- Time of year – As recorded in GIS.

3.4.1 Analysis Method

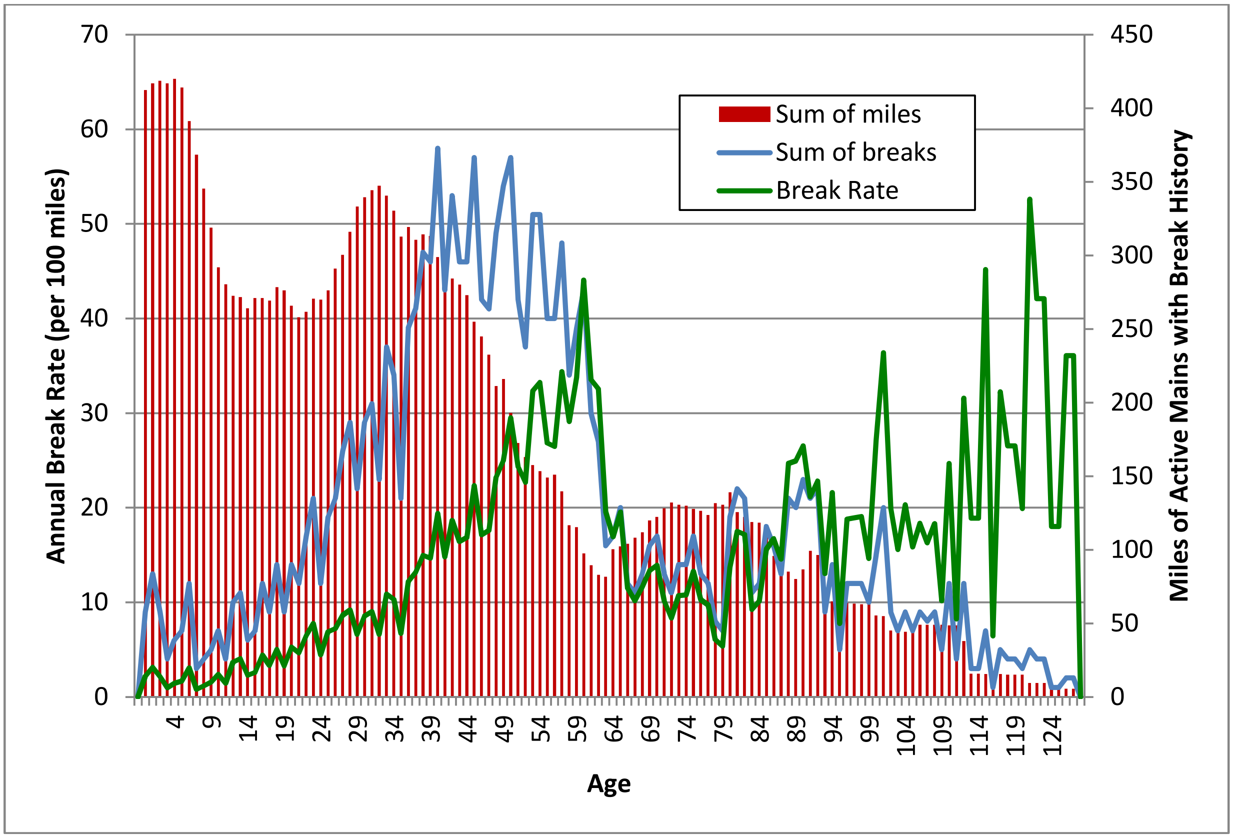
The break rate was calculated in terms of annual breaks per 100 miles of pipe. The break rate was calculated for each possible pipe age as:

Annual Break Rate = (100 * Number of Breaks) / (Miles of Main)

The break rate was determined by calculating the following:

- Number of breaks: Each break analyzed is associated to a pipe. A small subset of breaks that could not be associated to a pipe were excluded from this analysis. The age of the pipe at the time of the break was determined by subtracting the year of pipe installation from the year the pipe broke. For example, if a pipe was installed in 1980 and broke in 2000, the age of the pipe at the time of the break would be 20 years. If the same pipe broke again in 2010, the age of the pipe at the time of the second break would be 30 years. The number of breaks for a particular age was determined by counting all of the breaks that occurred at each pipe age.
- Miles of mains: LWS has relatively complete main break data since 1991. At the time of this analysis, the last complete year of break data available was for 2012. Therefore, this analysis determined the age of each currently active pipe in the system for each year between 1991 and 2012. Then, the cumulative miles of pipe for each age were calculated. For example, if a particular pipe was installed in 1980 and has been active through 2012, that pipe length was not to be included in the mileage for ages 0-10 because break data were not collected until this pipe was 11 years old (in 1991). The length was included in the mileage for ages 11-32 because break data was collected while the pipe was 11-32 years old.

Figure 3-9 shows the performance of the LWS system as pipes have aged. The break rate (shown in green) generally increases as the pipes increase in age. The Number of Breaks and the Break Rate are shown on the primary y-axis. The secondary y-axis displays the Miles of Main. Figure 3-9 also shows that a significant portion of the LWS system was installed in the more recent history with just over half of the system being less than 34 years old.



Notes:
The Break Rate (green line) displays the number of breaks on pipes of a given age per 100 miles of length.
The Sum of Breaks (blue line) displays the total number of breaks on pipes of a given age. The Sum of Miles (red bars) displays the total length of pipe that was ever that age that was included in the analysis.
It is important to note how the age of a pipe is calculated. The age is the year of analysis minus the installation data. For example, a pipe installed in 1900 would have an age of 91 in 1991, and an age of 101 in 2001. This pipe will have an age associated with it for each year in the analysis (1991-2012).
The results in the Figure 3-9 include the mileage of the active system only. Figure 3-9 includes lengths of Water Distribution Network (from GIS) that was provided by LWS (does not include lengths of GIS Production Network). LWS's Main Break Spreadsheet reports 1256 miles in the system.
Break data obtained from GIS database.

Figure 3-9 Summary of System Performance

For subsequent analyses in this section, the Miles of Main and Number of Breaks were grouped by potential deterioration factors to determine the extent to which each factor drove deterioration. Note, for each of the graphs in the subsequent sections, the age ranges were grouped such that at least 100 miles of pipe are included for each data point in a given asset class.

3.4.2 Deterioration by Material Quality

Based on industry experience, the rate of deterioration is impacted by the pipe material, manufacturing technique, manufacturing standards, general construction standards, and level of corrosion protection. For the purposes of this study, these factors are collectively referred to as Material Quality. Based on industry experience, local knowledge, and LWS system performance, four distinct Material Quality classifications were identified. Each pipe was associated with a classification based on its installation year. Each Material Quality classification is described below:

1. Vertical Pit Casting – Pipe installed between 1884 and 1933. Pipe in this era is generally characterized by relatively thick pipe walls.
2. Progressively Thinner Wall Spun Cast Iron - Pipe installed between 1934 and 1947. Pipe in this era was made progressively thinner and thinner as manufacturing methods improved. While this pipe had similar initial strength characteristics to Vertical Pit Casting, the thinner walls made it more susceptible to corrosion and a shorter life, particularly in the early to mid 1940s. The impact of World War II may also impact performance during this period as much of the steel and iron was being used for World War II, with the best material being shipped overseas (quality of material in U.S. was impacted).
3. Unprotected Ductile Iron and Thin Walled Cast Iron – Pipe installed between 1948 and 1972. Pipe in this era is characterized by thin pipe walls with cement mortar lining (CML) for internal corrosion protection. While this pipe had similar initial strength characteristics to Vertical Pit Casting, the thinner walls made it more susceptible to corrosion and a shorter life.
4. 1973-2012: Protected Ductile Iron and Polyvinyl Chloride (PVC) – The protected ductile iron has greater ductility than pipe from earlier eras (reduces consequence of pipe failure) and post-1973 generally included polyethylene (PE) encasement for external corrosion protection, making it much less susceptible to corrosion. While PVC does not corrode as it ages, small defects in the material can ultimate produce cracks as the pipe ages.

Table 3-3 presents the total length of each material in the system, percentage, and era of installation for each pipe material.

Table 3-3 Water Main Data by Material Quality

Material Type	Sum of Miles in Active System (miles)[2]	Approximate Percentage of Total	Approximate Era of Installation[1]
Vertical Pit Casting	195	16%	1884-1933
Progressively Thinner CI	46	4%	1934-1947
Unprotected Ductile Iron and Thin Walled Cast Iron	309	26%	1948-1972
Protected Ductile Iron and PVC[1]	635	54%	1973-Current
Grand Total	**1,185**	-	-

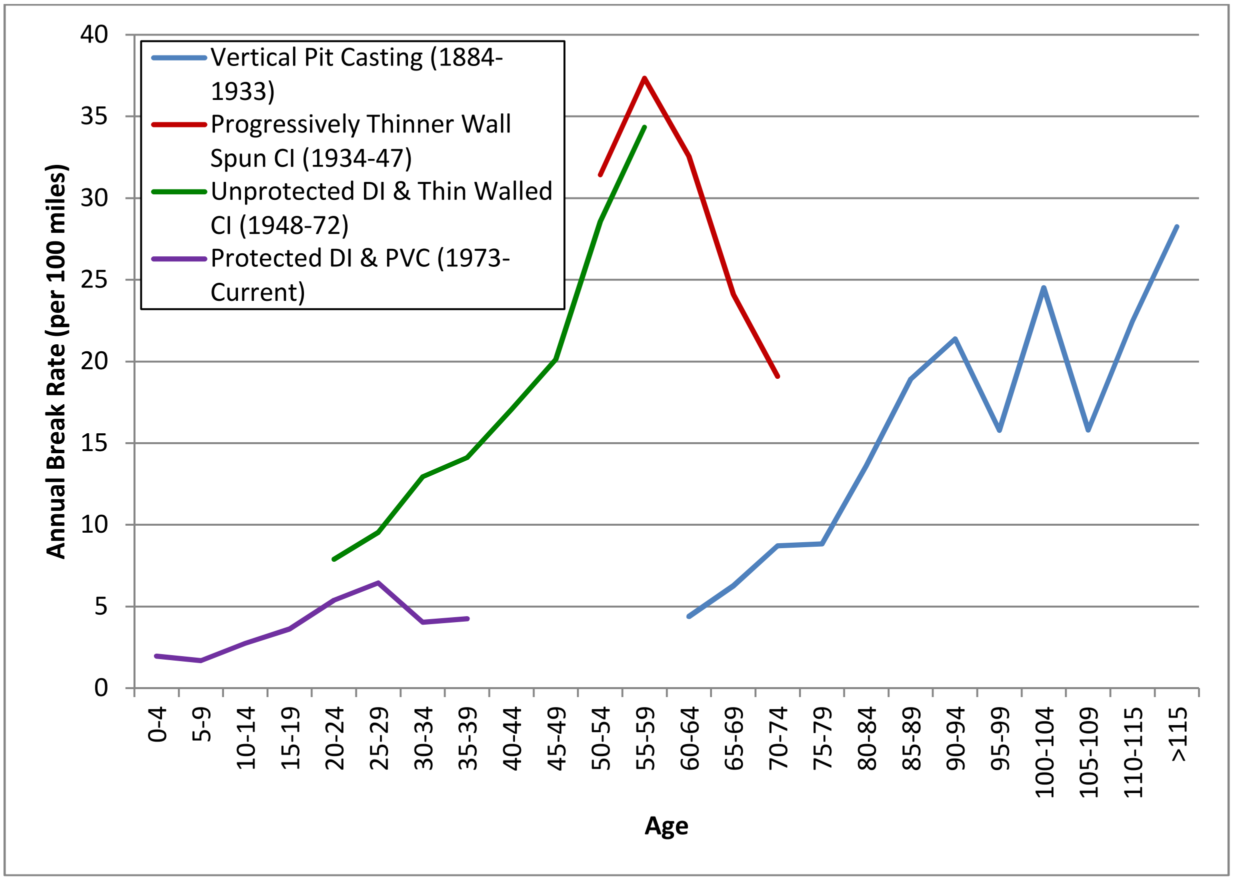
Notes:

1. *LWS staff noted that it was optional to include PE encasement from 1973 to 1978/1979. It became mandatory to include PE encasement on ductile iron pipe following 1978/1979.*
2. *The results in the Table 3-3 include the mileage of the active system only. Table 3-3 includes lengths of Water Distribution Network (from GIS) that was provided by LWS (does not include lengths of GIS Production Network). LWS's Main Break Spreadsheet reports 1256 miles in the system.*

Figure 3-10 displays the break rate versus pipe age for active pipe based on these four Material Quality asset classes. As presented, the youngest asset class, "Protected Ductile Iron and PVC" is performing significantly better than the "Unprotected Ductile Iron and Thin Walled Cast Iron" asset class. The difference in performance is dramatic where these asset classes overlap in age (20-39 years old) with the "Protected Ductile Iron and PVC" performing approximately twice as well as the "Unprotected Ductile Iron and Thin Walled Cast Iron".

The deterioration trend for the "Unprotected Ductile Iron and Thin Walled Cast Iron" is very steep and the overall performance is poor relative to the pipe age. A substantial peak is evident at 55-59 years of age, which is typical of the industry. In a recent study performed by the Utah State University Buried Structures Laboratory, *Water Main Break Rates in the USA and Canada: A Comprehensive Study*, (USU study) presents a similar peak failure age in the 41-60 year range for cast iron pipe (CI) and peak failure age for ductile iron pipe (DIP) pipe in the 21-40 year range. The theory here is that in the LWS system, once the pipe age exceeds 55-59 years in age, the installed pipes start to transition into the age where pipe walls were progressively thicker. These thicker pipes are less susceptible to corrosion and perform better. As demonstrated in the 2011 WaterRF Report, *Long-Term Performance of Ductile Iron Pipes,* a small difference in thickness can dramatically affect the time required for corrosion to penetrate a pipe wall. The downward trend shown in the "Progressively Thinner Wall Spun Cast Iron" asset class may occur because the thicker wall of the older materials is outweighing the natural deterioration of the pipe over time.

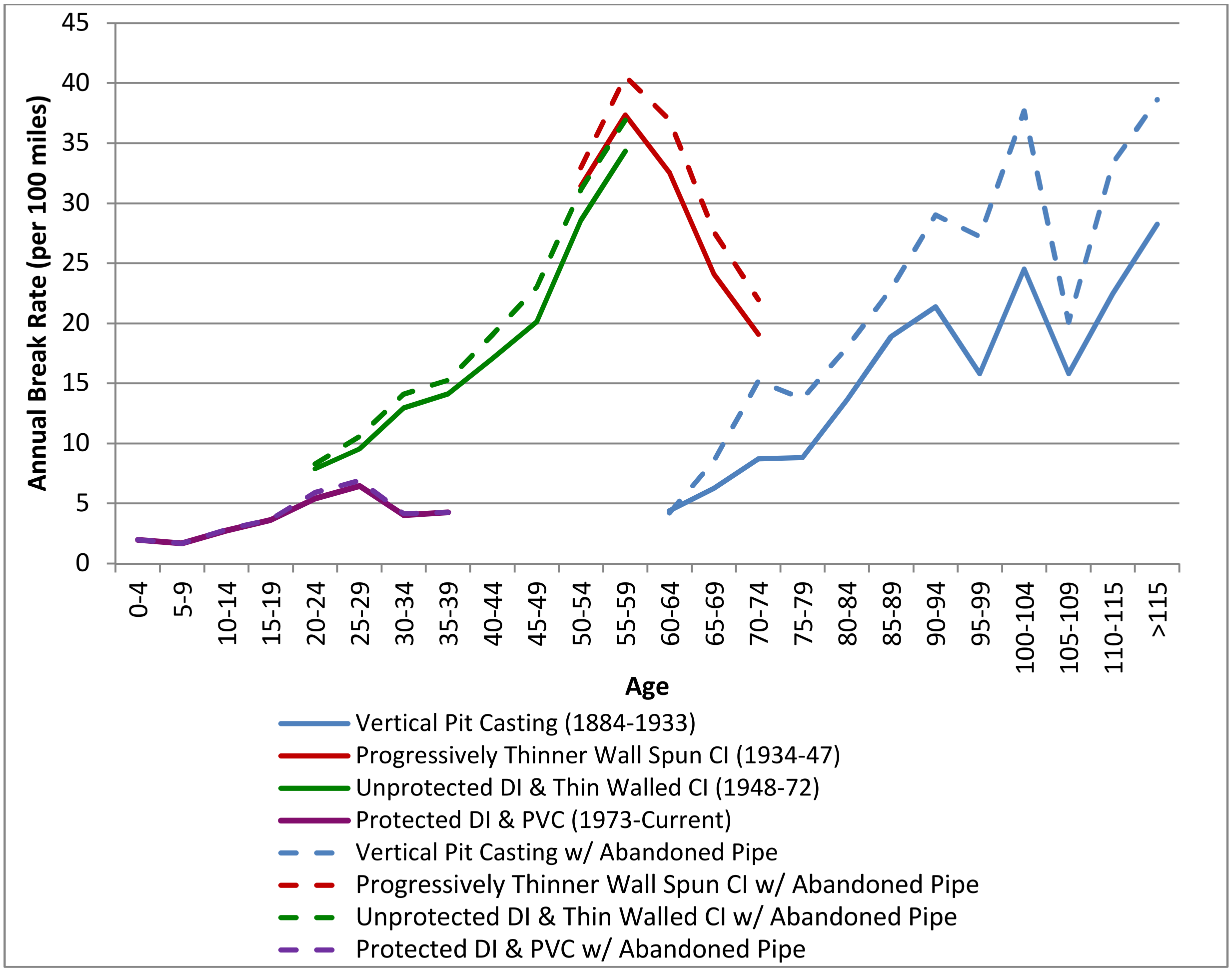
The oldest asset class, "Vertical Pit Casting", has a relatively low break rate for its age and is deteriorating at a more modest rate than the "Unprotected Ductile Iron and Thin Walled Cast Iron" asset class. In addition to thicker walls which prolong the life of this pipe, the analysis may also be showing the impact of LWS's replacement program as the weaker links in the system are more likely to have been replaced in the early years of the program, leaving behind pipes that relatively stable in respect to their environments.



Note:
Break count based on break data from GIS database.

Figure 3-10 System Deterioration by Material Quality

Figure 3-10 presents only the water mains that were active at the time of this study. To estimate the impact of historic documented LWS replacement projects, breaks and system mileage associated with Abandoned and Removed pipes were added. Figure 3-11 shows the break rate by Material Quality on the active system (solid lines) and on all pipes regardless of status (dashed line). This shows that historically LWS has made a conscious effort to replace older pipe with a high break rate. Recently, LWS has transitioned to place less emphasis on pipe age and more emphasis on break history and consequence of failure when prioritizing replacement projects which should flatten the steep deterioration trend in the Unprotected DI & Thin Walled CI and provide greater a level of service to the City's customers.



Note:

Break count based on break data from GIS database

Figure 3-11 System Performance by Pipe Material Based on Year of Installation

3.4.3 Deterioration by Pipe Diameter

Generally, break rates are inversely proportional to the diameter of the pipe. That is, smaller diameter pipe break sooner than large diameter pipe. This is because smaller diameter pipes generally have thinner pipe walls and smaller section modulus (i.e., less resistance to bending). For the purposes of this study, six diameter asset classes were identified. Table 3-4 presents the total length and percentage for each pipe size.

Table 3-4 Water Main Data by Pipe Diameter

Pipe Diameter	Number of Breaks[1]	Sum of Miles in Active System (miles)[2]	Percentage of Total Length in System
<6"	295	69	6%
6"	1740	603	51%
8-10"	178	105	9%
12"	168	161	14%
14-16"	117	116	10%
18-24"	81	59	5%
>24"	20[3]	72	6%
Grand Total	**2,599**	**1,185**	**-**

Notes:

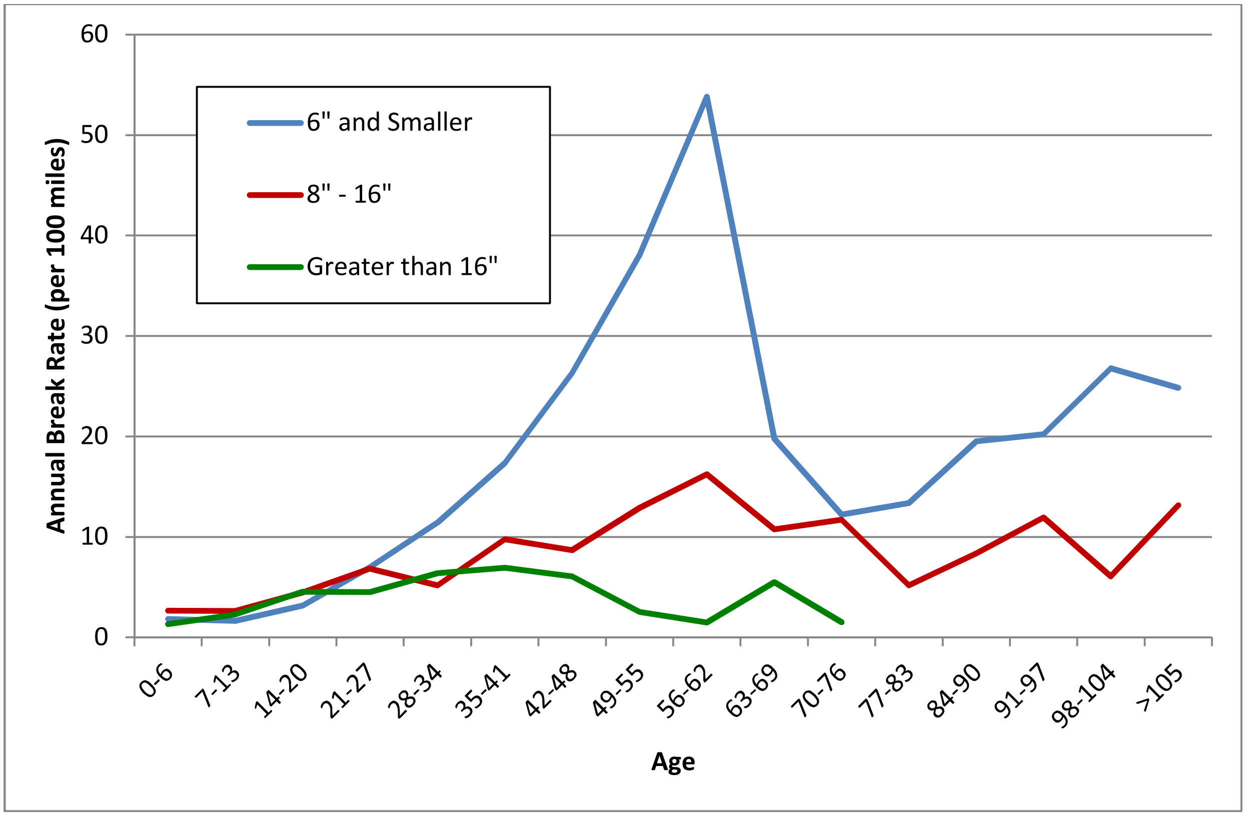
1. *Break total includes breaks on active system only.*
2. *The results in the Table 3-4 include the mileage of the active system only. Table 3-4 includes lengths of Water Distribution Network (from GIS) that was provided by LWS (does not include lengths of GIS Production Network). LWS's Main Break Spreadsheet reports 1256 miles in the system.*
3. *LWS has not experienced many breaks on mains greater than 24-inches in diameter. It is likely that the number of breaks on mains greater than 24-inches is less than shown in Table 3-4. See Section 7 for recommendations on data management techniques to minimize errors in associating breaks with the correct mains in the future.*

During analysis, the groupings were further consolidated into three asset classes as follows:

1. 6" and Smaller Pipe
2. 8"-16" Pipe
3. Greater than 16" Pipe

Figure 3-12 displays the system performance based on these three asset classes. The 6-inch and smaller asset class shows a higher break rate for the majority of the life of the pipe. This asset class also shows a strong peak at 50-70 years of age (similar to the Figure 3-10 in previous section).

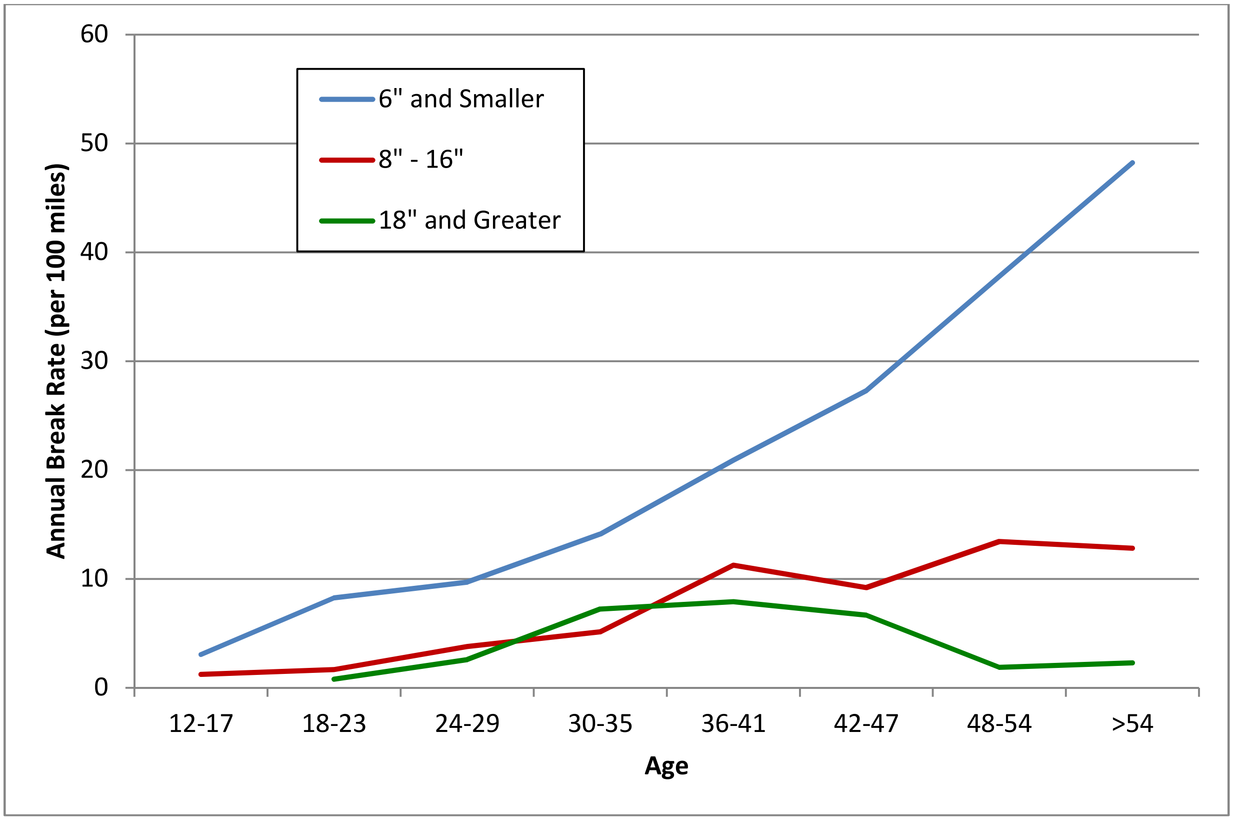
One item for consideration is that there is typically a higher tolerance for breaks on 6" and smaller pipe as compared to pipe greater than 16" due to the greater consequence of failure for the larger mains.



Notes:
LWS has not experienced many breaks on mains greater than 24-inches in diameter. It is likely that the number of breaks on mains greater than 16-inches (green line) is less than shown in Figure 3-12. See Section 7 for recommendations on data management techniques to minimize errors in associating breaks with the correct mains in the future.
Based on break data obtained from GIS database.

Figure 3-12 System Performance by Pipe Diameter

The data were also analyzed by combining characteristics in order to pinpoint the problem areas within the system. Figure 3-13 shows the Unprotected DIP and Thin Walled CI in all sizes and all ages. The Unprotected DIP and Thin Walled CI asset class is degrading very quickly in pipes 6" and smaller, as shown by the blue line in Figure 3-13. The break rate is nearly 50 breaks per 100 miles for pipes above 54 years in age. This graph shows that resources focused on this asset class would likely have a strong return on investment.



Note:
Based on break data obtained from GIS database.

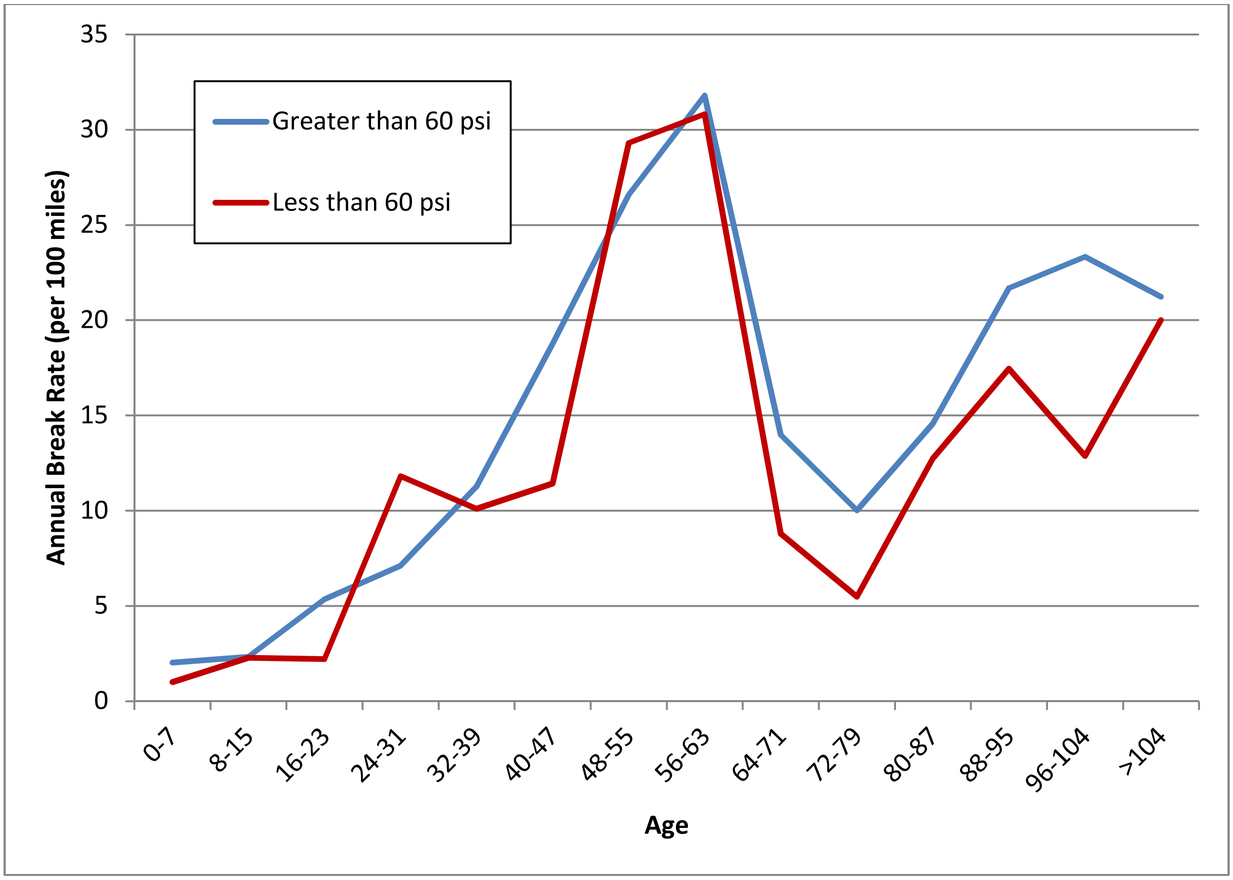
Figure 3-13 System Performance by Pipe Size for Unprotected Ductile Iron Pipe and Thin Walled Cast Iron Pipe

3.4.4 Deterioration by Operating Pressure

Generally, pipes exposed to higher static operating pressures break at an earlier age due to higher hoop stresses. For the purposes of this study, four operating pressure asset classes were initially identified as follows:

- Less than 60 psi
- 60-75 psi
- 75-90 psi
- Greater than 90 psi

During analysis, the groupings were further consolidated into two asset classes, above 60 psi and below 60 psi. Figure 3-14 displays the results of this analysis. It is difficult to identify any trend by pressure over time. The average break rate for pipes with pressures less than 60 psi is lower than pipes with pressures above 60 psi, but this varies with age, and the trend is very weak. Therefore, it is not believed that static pressure is a significant driver for main breaks in the LWS system.



Note:
Based on break data obtained from GIS database.

Figure 3-14 System Performance by Pressure

3.4.5 Deterioration by Slope

Often in hilly areas, the ground will creep at a slow rate. The gradual slope creep can apply additional stresses on pipes installed in steep slopes causing them to break sooner. Main breaks were examined based on slope. Figure 3-15 displays the break rate of pipes based on slope. On average, pipes with higher slopes do fail at higher rates, but there is no strong trend identified in this analysis. Overall, Lincoln is relatively flat. Strong trends identified in other cities typically have significant quantities of infrastructure with ground slopes of 5 percent or greater, which are not common within Lincoln. Therefore, it is not believed that ground slope is a primary driver for main breaks in the LWS system.

Note:
Based on break data obtained from GIS database.

Figure 3-15 System Performance by Slope

3.4.6 Deterioration by Soil Corrosion

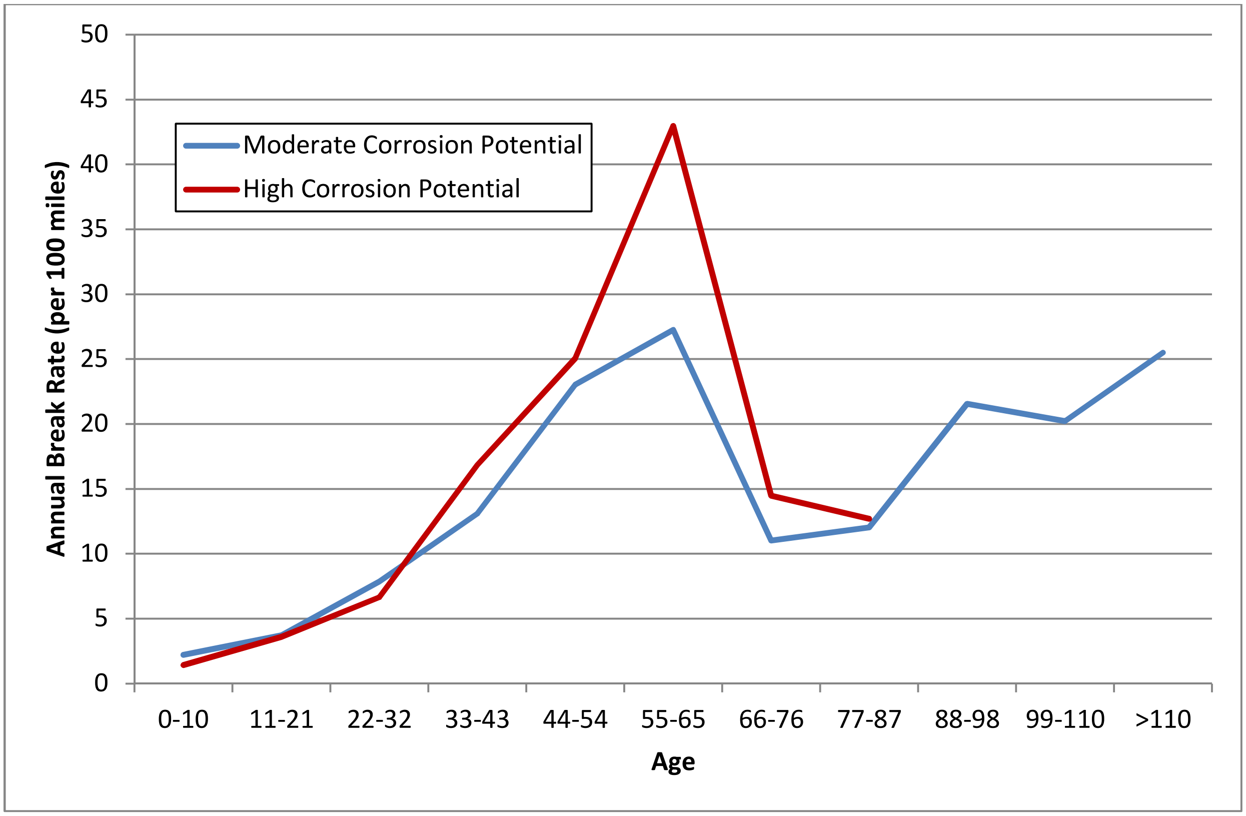
Corrosion rates vary significantly due to soil type and can significantly affect a pipe's lifespan. External pitting from corrosive soils is a known problem on metallic pipe materials including grey cast iron, ductile iron and steel. The rate of external pitting attack on unprotected ferrous materials is governed primarily by the corrosivity of the environment and the presence or lack of corrosion protection. Therefore, soil type can be considered as an influence on main breaks.

Several factors in the soil can affect its corrosivity potential, including pH, resistivity, moisture content, oxidation-reduction (redox) potential, organic and sulfides content. Soils of low (<4) pH serve well as an electrolyte and soils of high (>8.5) pH are often high in dissolved salts, both of which can be corrosive to metallic pipe. Soils of lower resistivity (>2000 ohms), are likely to cause more rapid pitting attack to ferrous materials at rates that increase as resistivity decreases. Prevailing moisture content due to water tables is extremely important to soil corrosion. The redox potential of a soil is significant, because the most common sulfate-reducing bacteria can live only in anaerobic conditions. Sulfide determination is important since sulfate-reducing bacteria (SRB) can be a primary cause for the acceleration of the cathodic reaction which leads to corrosion.

In addition, corrosion as graphitization is a major factor influencing iron pipe failure. Graphitization of grey cast irons can be expected when soil conditions favor anaerobic bacterial growth, with the appropriate conditions of pH, dissolved salts, and organic content. The result is a matrix consisting of a mass of residual graphite flakes interspersed with oxides of iron, which are the graphite-containing corrosion products. This material matrix leads to corrosion-induced loss of wall thickness which can eventually lead to pipe failure. Additional external corrosion can be caused by galvanic corrosion from dissimilar metals (commonly with copper services) and stray electrical currents.

Soil information was obtained through the Web Soil Survey service from the Natural Resources Conservation Service (NRCS) of the U.S. Department of Agriculture. Soils are classified in GIS by the NRCS as having either a high, moderate or low potential for steel corrosion. The vast majority of the soils in Lincoln have either moderate or high corrosion potential (very little data for low corrosion soils). Due to the limited amount of pipe located in low corrosion soils (less than 100 miles of data for all age ranges), the low corrosion asset class was excluded from the analysis.

Figure 3-16 displays the break rate of the LWS system by age. There appears to be a slight trend towards higher break rates in high corrosion potential versus moderate corrosion potential, though the trend is not strong. While the study team believes corrosion is a primary driver for deterioration rates, it is believed that existing readily available data at the time of this study was not sufficient to identify where these highly corrosive soils exist.



Note:
Based on break data obtained from GIS database.

Figure 3-16 System Performance by Soil Corrosion Potential

Figure 3-17 displays a map of the soil corrosion potential. It is evident from the figure that the majority of the City is covered with soils with moderate to high corrosion potentials. It is probable that the vast majority of the LWS system is covered with soils that have a relatively high corrosion potentials, which is likely one of the reasons that a stronger trend was not identified in Figure 3-17. These soils with relatively high corrosion potentials reinforce the need for proper material selection, external corrosion protection for DIP, and cathodic protection of critical lines.

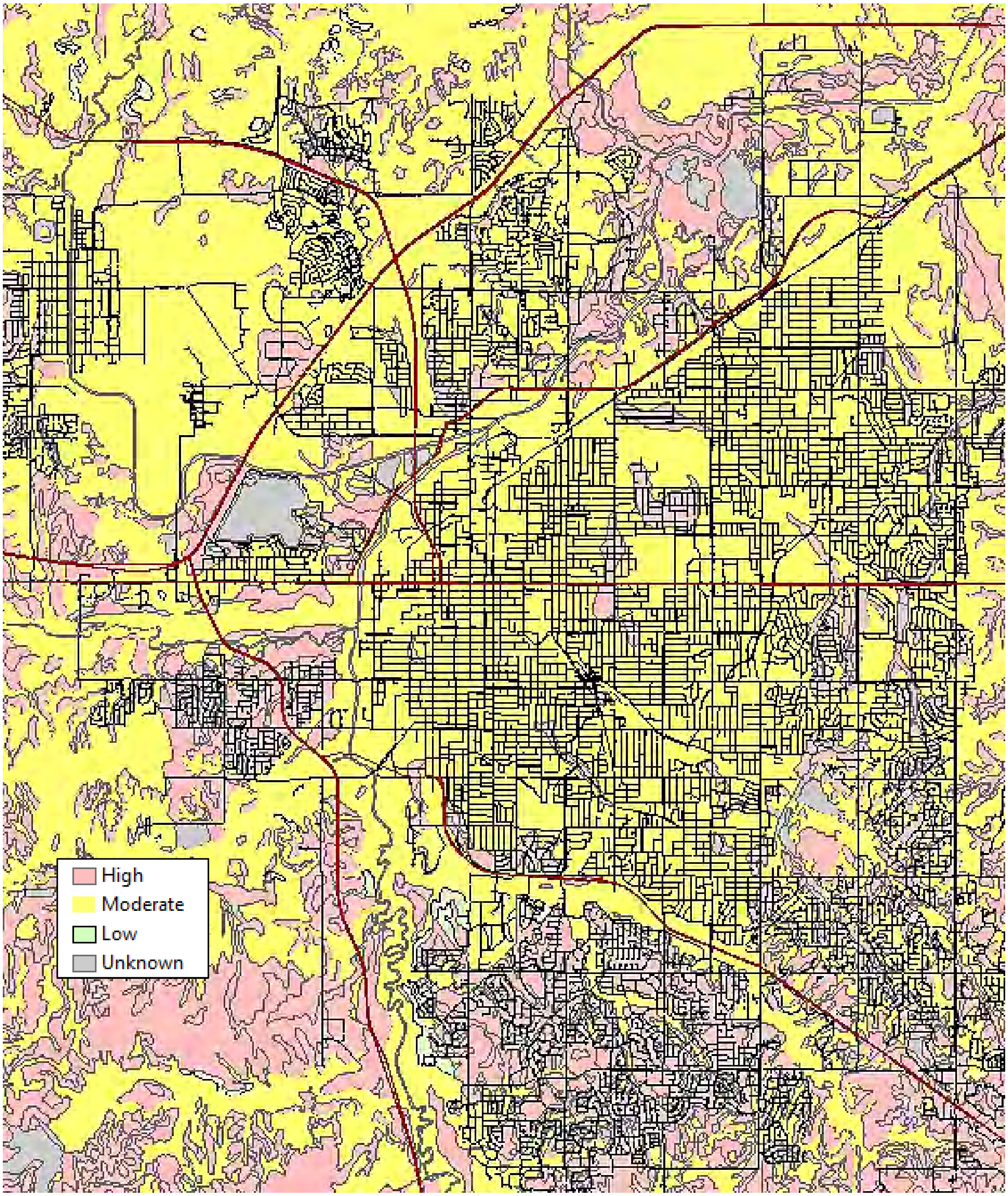


Figure 3-17 Soil Corrosion Potential Map

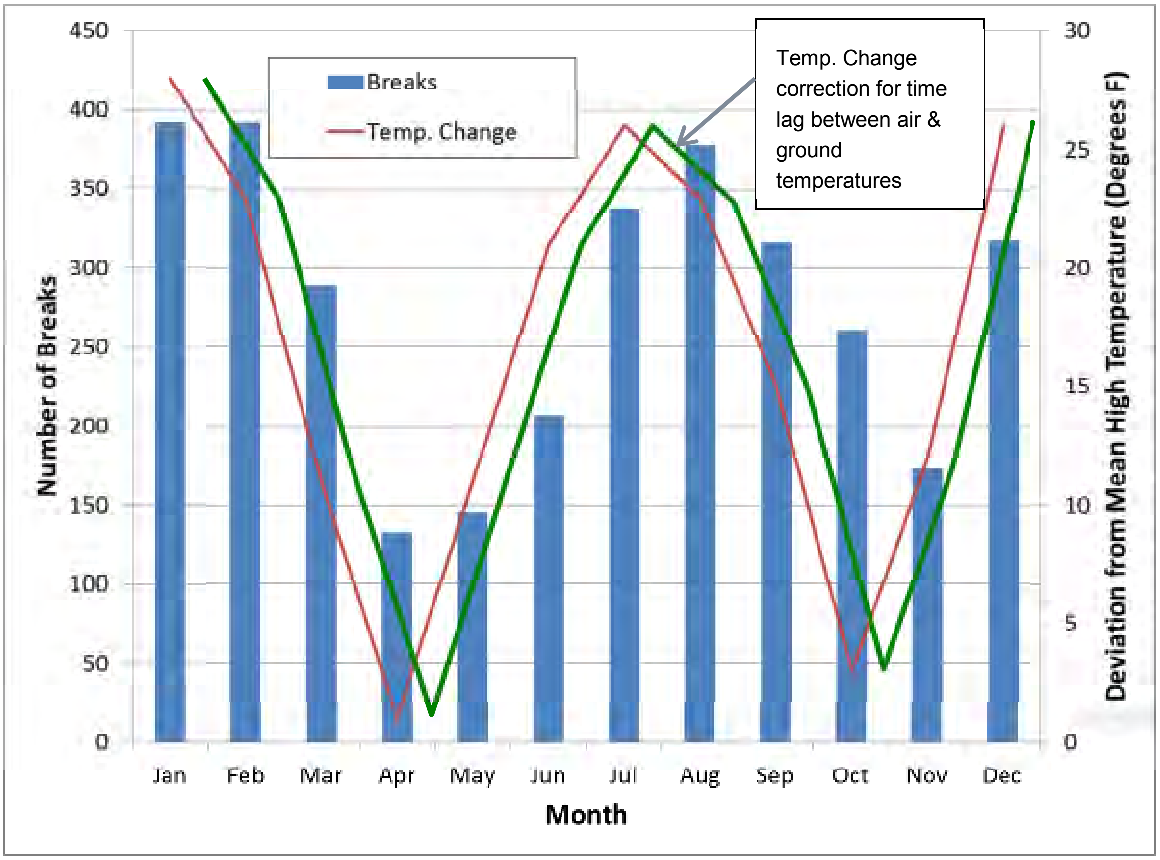
3.5 Performance by Time of Year

Main breaks were analyzed for seasonal variations in temperature and precipitation. Figure 3-18 displays the total main breaks from 1991 through 2012 by month and the deviation from average high temperature for each given month in Lincoln. It is clear from the figure that the majority of breaks occur when the temperature deviations are highest (i.e., temperatures are at the high and low temperature extremes); with fewer breaks occurring when the temperature deviations are small (i.e., during mild months).

Breaks are caused by a combination of deterioration, strain accumulation, and fatigue. The seasonal variations are trigger events. When the weather turns cold, it is common to see a spike in breaks on CI. This is due to the colder water entering the pipes which cause thermal stresses. Frost heave can also play a role in causing increased number of breaks in cold, winter months.

In the summer, the increased breaks could be caused by soil shrinkage as clay soils dry. Demand is also higher in the summer which results in higher velocities in the pipes. The increased velocities are a potential cause in the increase in breaks in the summer. Small pressure transients from numerous sprinkler systems shutting off can also cause an increase in breaks.

One item to note is that the ground/water temperature will likely lag air temperature by a few weeks. If the air temperature deviation (red line) is lagged two weeks to estimate ground/water temperature (green line), the correlation between temperature deviations and breaks is even stronger.



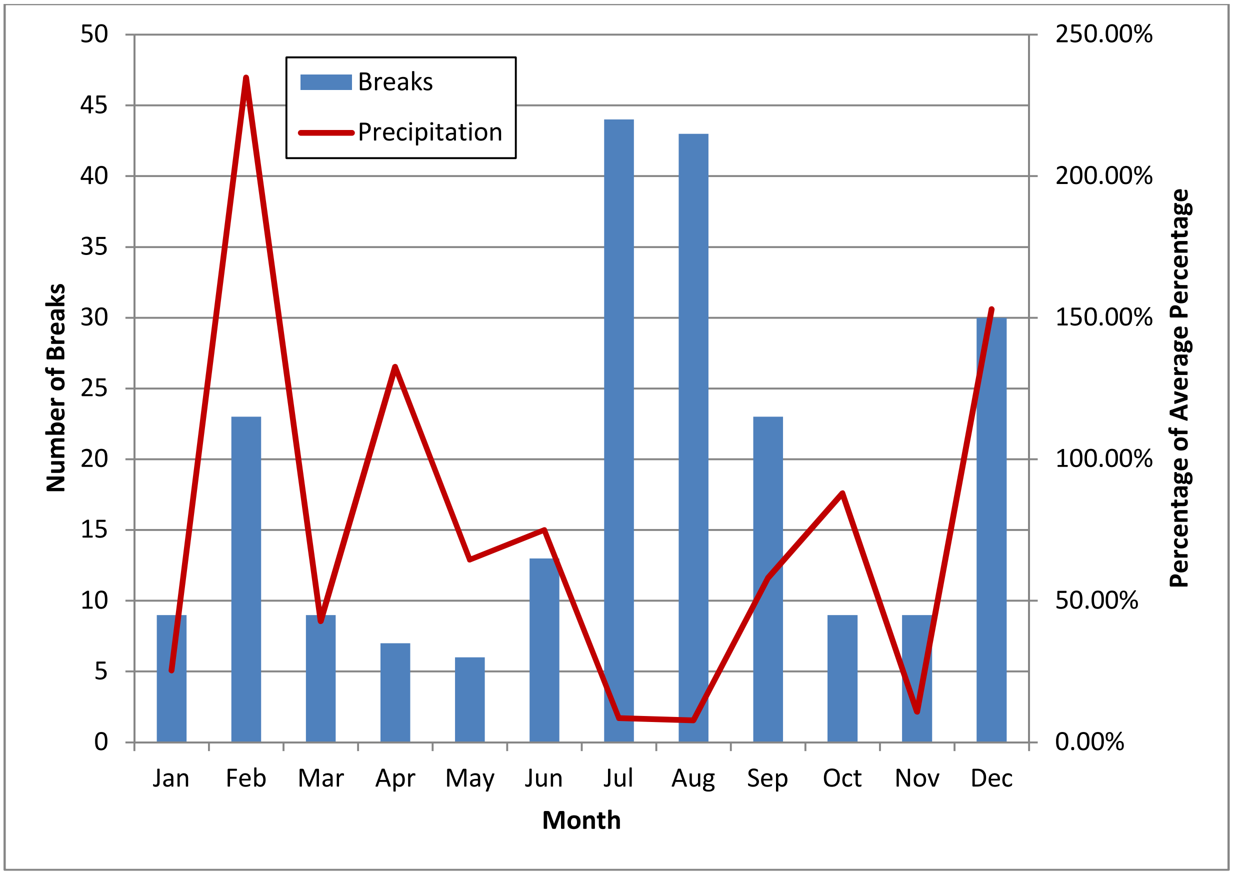
Notes:
Average temperature for each month was taken from The Weather Channel (weather.com). The green line displays an approximated 2-week lag between the change in air temperature and the ground temperature. Based on break data obtained from GIS database.

Figure 3-18 System Performance by Time of Year

3.6 Performance Compared to Annual Precipitation

System performance can also be impacted by amount of precipitation in a given year. Relatively dry years tend to show increased numbers of breaks as compared to years of average precipitation. Dry weather causes soil shrinkage leading to beam bending which causes water main breaks. Other potential causes of increased breaks include higher water main pressures and velocities due to increased water usage in dry weather.

Figure 3-19 displays the water main breaks in 2012 (which was a relatively dry year for LWS). Other utilities in the region also experienced higher than average break counts in 2012. Typically, June and July are the months with the largest precipitation and the largest water demands in Nebraska. In 2012, there was very little precipitation during this period in Lincoln, which happens to coincide with the largest amounts of breaks (see Figure 3-19).



Note:
Based on break data obtained from GIS database.

Figure 3-19 System Performance by Precipitation

4.0 Useful Life Forecast

The useful life of a water main represents the median number of years the water main is expected to remain in service from installation to replacement. Useful life estimates are meant for planning purposes only. The life of a particular pipe is highly dependent on the quality of construction, the quality of the manufacturing, the pipe's unique environment, and other pipe specific characteristics, all of which are eventually reflected in its break history or discernible through various direct assessment techniques.

In the US, life expectancies of water mains typically range anywhere from 50 years to 300 years. This broad range is due to a variability of installed conditions, system operations, and pipe manufacturing and materials. As a result, pipe age by itself is a poor predictor of pipe condition. In addition, the "failure" of a pipe is not a definitive event. Unlike a person, a water pipe can be made to last indefinitely, as long as a utility is willing to repair it. Different utilities choose to manage their systems differently, and as a result, have different life spans for their pipes.

For the purposes of this study, three methods were used to estimate water main useful life:

- Asset class deterioration projection
- Survival Model (i.e., Weibull Model)
- Benchmarking

The results of these methods will be compared to determine a range of realistic useful life estimates that will be the foundation for the identification of a fiscally sustainable reinvestment level estimate.

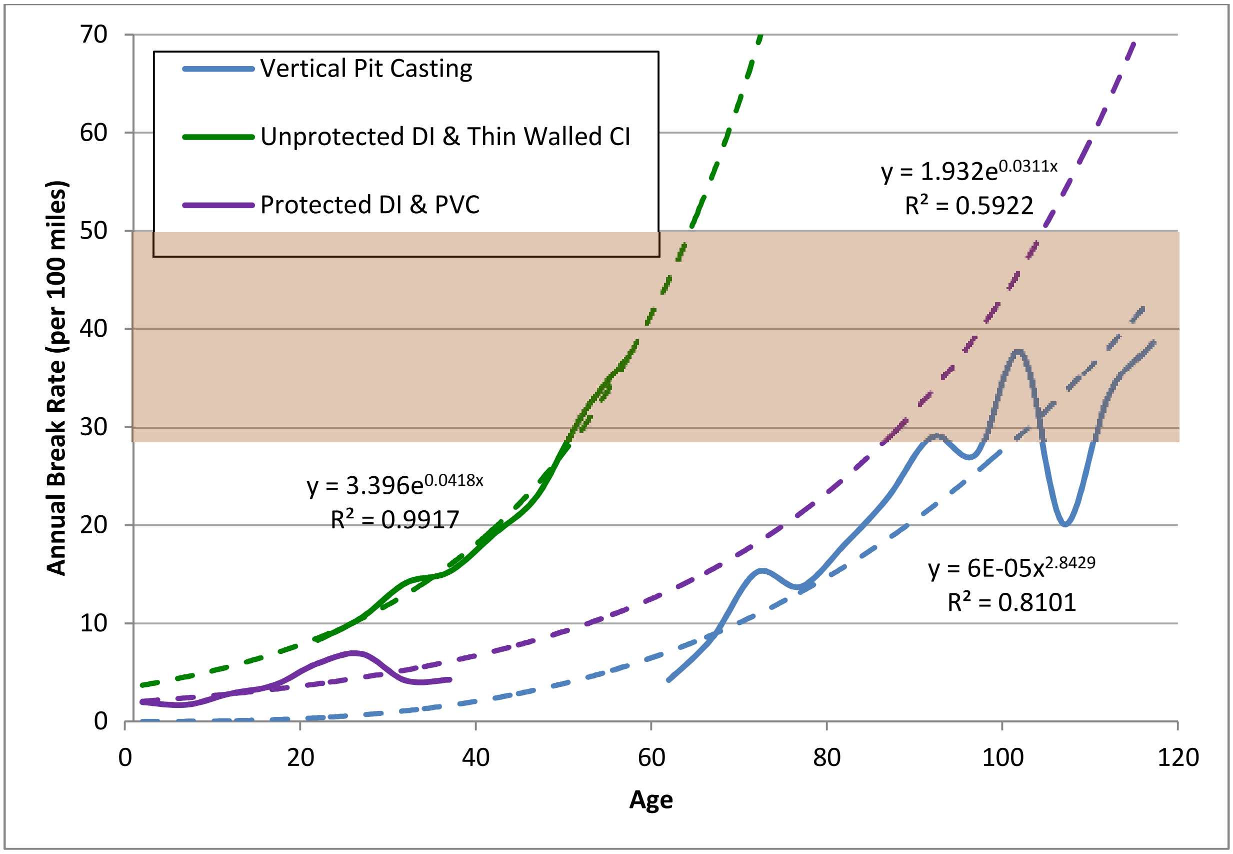
4.1 Asset Class Deterioration Projection

In this analysis, the data was broken into the same asset classes as Section 3.4.2 which include:

1. Vertical Pit Cast Iron
2. Progressively Thinner Wall Spun Cast Iron
3. Unprotected Ductile Iron and Thin Walled Cast Iron
4. Protected Ductile Iron and PVC

Because the Progressively Thinner Wall Spun Cast Iron asset class does not show a realistic trend (trending downward), it was excluded from this projection. It was assumed that the younger half of this asset class will trend similar to the Unprotected Ductile Iron and Thin Walled Cast Iron asset class (green line) and the older half of the asset class will trend similar to the Vertical Pit Casting Asset Class (blue line). For each of the three remaining asset classes, trend lines were fitted to the data set to allow for projections into the future.

Figure 4-1 shows the output of the analysis for each asset class. The Break Rate is on the y-axis and the pipe age is on the x-axis. The red band represents the target performance range for this asset class where pipes would be replaced. This range differs from the system-wide break rate goal, because the intent is to replace the poor performing assets so that the overall system performs in the 14 breaks per 100 miles range. The minimum and maximum service life is then determined by calculating when the Break Rate trend enters and exits the target performance range.



Note:
Based on break data obtained from GIS database.

Figure 4-1 Useful Life Projections Based on Pipe Material

Table 4-1 presents the useful life of the three asset classes based on various definitions of failure that could be assumed. For example, if one was to assume a definition of failure of 20 annual breaks per 100 miles, the vertical pit casting asset class would have a service life of 88 years.

Table 4-1 Useful Life Projections Based on Pipe Material

	Median Useful Life, years		
Break Rate (Breaks per 100 miles)	Unprotected DI & Thin Walled CI	Protected DI & PVC	Vertical Pit Casting CI
30	52	88	101
40	59	97	112
50	64	105	121

It is important to note that the vertical pit casting curve (blue line) may have been flattened over time with replacements before the break data started to be tracked in 1991. This unknown may result in an over prediction of the useful life for the vertical pit casting asset class.

With limited data on the Protected Ductile Iron and PVC asset class, it is difficult to estimate the useful life until more history is available (which provides more data for curve fit). In review of the data, there were several instances where a break was tied to a newer asset class due to close proximity to the break when the break should have been tied to the abandoned/replaced main. This was corrected when found, but it is likely that the actual break rate for the Protected Ductile Iron and PVC asset class is slightly lower than what is shown in Figure 4-1 above. It should also be noted that the R-squared value for the Protected Ductile Iron and PVC asset class is lower than the other two asset classes. It is difficult to project the useful life of this asset class with the few years of data that are available at this time.

4.2 Survival Model (i.e., Weibull Analysis) Results

4.2.1 Weibull Distribution Analysis

The Weibull Distribution is the most commonly used methodology to assess service life in the water industry. The following section describes the assumptions and results of the Weibull analysis performed for LWS. For the Weibull Distribution Analysis, the Isolated Pipe definition defined in Section 3.2.3 was used.

4.2.2 Definition of Failure

The definition of failure is a critical component in this analysis. A pipe “failure” is meant to describe when a pipe asset should be replaced. Note, this is a planning level definition of failure that is meant to describe the average break-rate failure for the entire system. Other failure definitions may derive from direct condition assessment, water quality issues, or capacity concerns. However, the latter definitions are hard to quantify and evaluate, and therefore are frequently not included in condition assessment analyzes

It is assumed that the actual definition of failure for an asset class or a pipe will vary due to its relative risk and associated cost to rectify. For the purpose of this analysis, a pipe failure has been defined as three or more breaks occurring on an Isolated Pipe.

4.2.3 Summary of Model Input Data

The break database includes breaks dating to 1984. However, based on LWS staff input, it is believed that there were a substantial number of breaks missing between 1984 and 1990. It should be noted that these missing data will generally skew the results towards a longer useful life than would occur if the missing data were included in the analysis.

Isolated Pipes less than 100ft in length were not modeled because failures in these pipes are rare and could skew the results from a Capital Improvements Program (CIP) perspective. Isolated Pipes less than 100 ft in length comprise less than 5 percent of the LWS distribution system by length. Based on the criteria above, 12,224 Isolated Pipes were analyzed, of which 363 failed (meaning they had 3 or more breaks) and 11,861 passed. A map of these pipes is included in Figure 4-2 below.

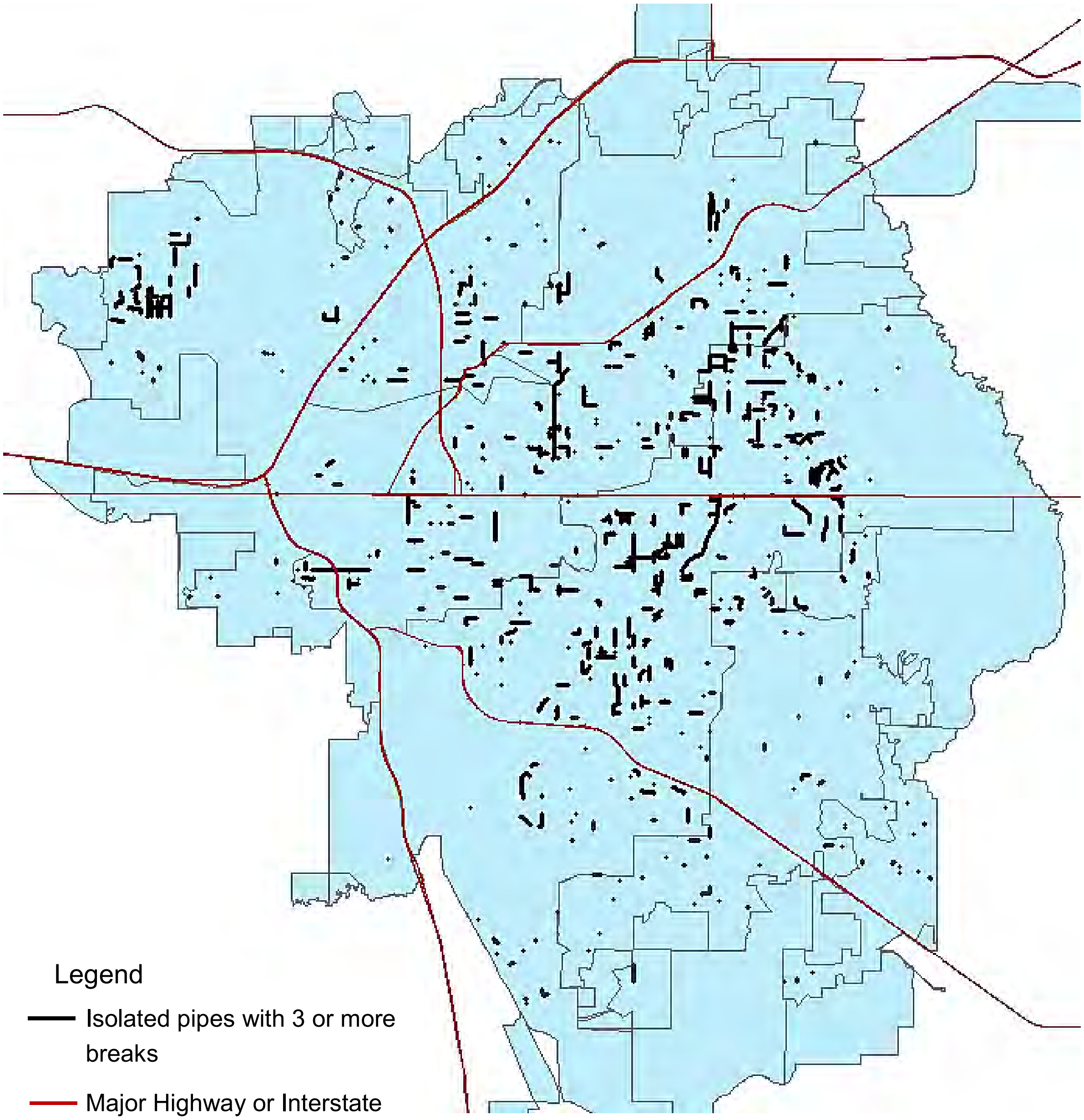


Figure 4-2 Pipe Network Map

The initial analysis used all failure data in one model and assumed that each pipe has an equal probability of failure regardless of install year, material, diameter or other pipe attribute.

Subsequently, the failure data was parsed by separating the pipe into eras of similar Material Quality. These eras were the following (descriptions are included in Section 3.4.2):

- 1884 – 1933 (presumably Vertical Pit CI)
- 1934 – 1947 (presumably Progressively Thinner Wall Spun CI)
- 1948 – 1972 (presumably Unprotected Ductile and Thin Walled CI)
- 1973 – 2012 (presumably Protected Ductile Iron and PVC pipe)

The failure data for each era was analyzed separately, using the number of failures and corresponding remaining unfailed pipe for that era.

4.2.4 Model Methodology

A review of the assumptions, output needs, and data were reviewed. Based on this review, a Weibull distribution was recommended to fit its reliability model. This type of model is often used in industrial fields to predict time to failure. The Weibull probability density function (pdf) is shown in Equation 1 below. The Weibull pdf distribution is a parametric function. Its shape is determined by three parameters: β (beta) which determines the shape of the distribution (for example, is it bell curved or not), η (eta) which determines scale (linked to the units of time used in the model and identifies the characteristic life at what value 63.2 percent of the units will fail) and γ (gamma) which is a location value. Γ represents the unit of time before failures can occur and has the same unit used for time t.

Equation 1: $$f(t|\beta,\eta,\gamma) = \frac{\beta}{\eta}\left(\frac{t-\gamma}{\eta}\right)^{(\beta-1)} e^{-\left(\frac{t-\gamma}{\eta}\right)^{\beta}}, \beta > 0, \eta > 0, -\infty < \gamma \; and \; t < \infty.$$

The Weibull cumulative density function (cdf) is a more useful expression. The value of the cdf at time t is equal to the area under the pdf up to time t. The Weibull cdf defines the Weibull unreliability function, or probability of failure. This function represents the probability of an item failing by time t. This equation is shown if Equation 2.

Equation 2: $$F(t|\beta,\eta,\gamma) = 1 - e^{-\left(\frac{t-\gamma}{\eta}\right)^{\beta}}, \beta > 0, \eta > 0, -\infty < \gamma \; and \; t < \infty.$$

To estimate the parameters for the model, the installation date of each pipe was leveraged as well as information pertaining to a failure date prior to the end of the study period. The study period for HDR's preliminary reliability model is 1984 until 2012. All selected pipes in the system on and between these dates were reviewed for breaks. If any of the pipes met the criteria for failure, the date of the third leak (for pipes greater than 100 ft in length) was recorded. Pipes as of the end of 2012 that had not yet met the failure criteria were treated as censored observations with the assumption that they still could fail at any point after 2012.

4.2.5 Model Output

HDR used the software package Weibull++ 7 (Reliasoft)1 to fit the Weibull distribution to the data. Based on the rate of pipes surviving over time, the software fits a distribution curve to model the underlying distribution which can best explain the observed failures (i.e., the 363 failed Isolated Pipes in a system of 12,224 Isolated Pipes).

4.2.6 Summary of Weibull Model Results

Figure 4-3 displays the Weibull Cumulative Distribution Function. Based on the data available, the model estimates that the true median time of failure (i.e. service useful life) is:

- 230 years for Vertical Pit CI pipe
- 60 years for Unprotected Ductile and Thin Walled CI pipe
- 140 years for Protected Ductile Iron and PVC pipe

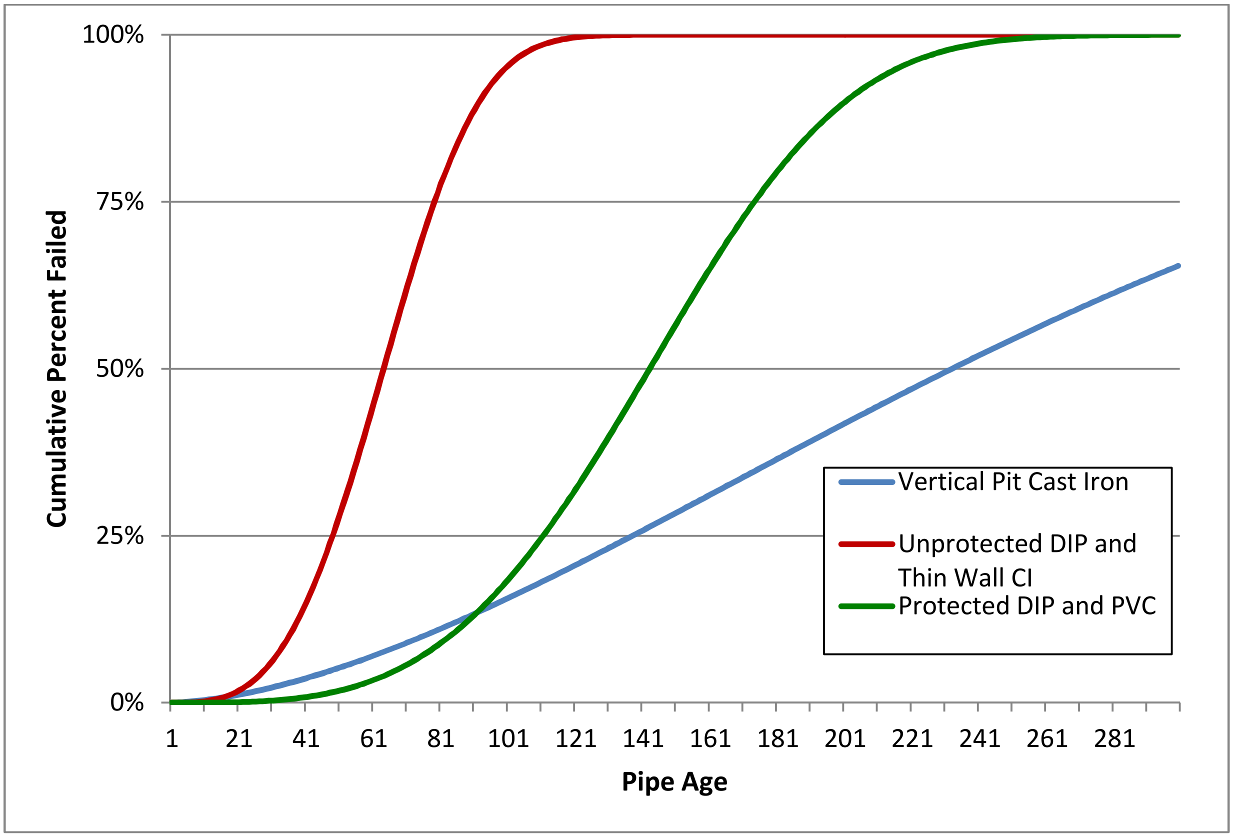


Figure 4-3 Weibull Cumulative Distribution Function

[1] Reliasoft Corporation. 2006. Weibull++: Life Data Analysis (Weibull Analysis) Software Tool. Reliasoft Corporation, Tucson, AZ. http://www.reliasoft.com/Weibull/index.htm.

When using a model like this to forecast long-term CIP levels, it is important to consider that this model has certain limitations, including:

- Based on the definition of failure, only 3 percent of assets have failed.
- By count, approximately 52 percent of the Isolated Pipe modeled was built prior to 1984 when readily available break history was first collected (this is anticipated to bias the model to a slightly higher useful life).
- The average age of the system modeled is only 41 years.
- This model does not account for variations in the failure by other secondary characteristics that influence performance (pressure, diameter, soil corrosivity, etc.).
- As more data is collected over time, the accuracy of the model will increase.
- Breaks that are widely separated in time may be random events rather than caused by a common aging process (i.e., corrosion). It may be useful to further investigate how prevalent this is.

4.3 System Benchmarking

As a comparison, data from the USU Study[2] was used along with values reported by the Water Research Foundation in recent reports. The USU Study had 188 responses from utilities in the U.S. and Canada which included data on 117,603 miles of water main. The USU Study represents approximately 10 percent of the water mains in service in the U.S. today. The USU Study asked for the most recent 12 months of data which correlates to the year 2010. The AWWA also conducts periodic surveys to establish benchmarks. The AWWA surveys collect data on 34 metrics that cover the following areas: Organizational Development, Customer Relations, Business Operations, Water Operations, and Wastewater Operations. Of these 34 metrics, 11 are associated with the Water Operations area. The metric of concern when discussing the Main Replacement Program is Water Distribution System Integrity, a measure of the water main breaks. The AWWA Benchmarking Survey uses leaks and breaks associated with valves, hydrants, and service connections in addition to the pipe itself. The analysis presented in this Master Plan used only breaks associated with the pipe and did not include appurtenances, therefor, the AWWA Benchmarking data will not be used as a comparison to LWS.

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[2] *Water Main Break Rates in the USA and Canada: A Comprehensive Study*

4.3.1 Material Comparison

A direct comparison is not possible due to variations in grouping of the data between the USU study and LWS; however, it does provide a good benchmark between LWS and the rest of the country. Table 4-2 shows the percentage of total pipe length for various pipe materials from the USU Survey. The regional results include North Dakota, South Dakota, Nebraska, Iowa, Kansas, and Missouri.

Table 4-2 Material Comparisons

	Nationwide (% of total miles)	Regional (% of total miles)	LWS[1] (% of total miles)
Cast Iron	29	36	43
Ductile Iron	28	26	30
PVC	23	34	27
Other	20	4	0.1

Note:

1. *The percentages shown in the Table 4-2 for LWS include the mileage of the active system only. Table 4-2 includes lengths of Water Distribution Network (from GIS) that was provided by LWS (does not include lengths of GIS Production Network). LWS's Main Break Spreadsheet reports 1256 miles in the system.*

LWS has a fairly typical mix of pipe material in its distribution system.

4.3.2 Useful Life

Neither the USU Study nor the AWWA survey contained data on the expected useful life of various pipe materials. The AWWA report *Buried No Longer: Confronting America's Water Infrastructure Challenge* presents a range of useful life for various pipe materials. In this report a large system is defined as a system that serves more than 50,000 people. Therefore, LWS is a large system. Table 4-3 presents the estimated useful life for Midwest Large systems from the AWWA report. The ranges represent the extremes of benign ground conditions and skilled installation practices corresponding to long life and harsh ground conditions and unskilled installation methods corresponding to short life. The projected service lives of the various materials examined in this Master Plan correspond well to the data presented by the AWWA report.

Table 4-3 Useful Life Comparison

Pipe Material	Benchmarking Service Life (from AWWA) (years)	Asset Deterioration Projection (years)	Survival Model Results (years)
Cast Iron	125	101-121	230
Cast Iron, Cement Lined	85-120	NA	NA
Ductile Iron	50-110	52-64	60
Protected Ductile Iron	NA	88-105	140
PVC[1]	55	88-105	140

Note:

1. The Useful Life projections shown for PVC pipe vary significantly, predominantly because limited data are available on PVC as it has not been in use as long as some of the other materials.

5.0 50-Year Renewal Forecast

The purpose of this analysis is to establish a 50-year renewal forecast.

5.1 Assumptions

The development of the long term CIP forecast for water main replacement was based on the following assumptions:

- The three Weibull curves as documented in Section 4.2.5 were used.
- Progressively Thinner Walled CI installed between 1934 and 1940 will deteriorate similar to Vertical Pit Cast pipe and the Progressively Thinner Walled CI installed between 1941 and 1947 will deteriorate similar to Unprotected Thin Walled DI & CI.
- LWS would like to maintain its current level of service and maintain a break rate at or below 14 breaks per 100 miles of pipe.

5.2 Method of Calculation

The Weibull Distribution predicts the cumulative percent of the system that will fail as the pipe ages. A summary of the three distributions used is included in Figure 4-3. For each year in the 50-year CIP forecast, the system mileage was categorized by installation era and by pipe age. This mileage was then multiplied by the corresponding cumulative percent failed as projected by the appropriate Weibull distribution to determine the cumulative miles of pipe that the model would predict would have failed.

The model predicted that 152 miles should have failed system wide by 2014. Because LWS has had a proactive water main replacement program for several years, some of this failed pipe has already been replaced. An analysis was conducted which determined that 48 miles of active pipe has failed based on the definition of failure used in this study. A map of these pipes is shown in Figure 6-1. Therefore, it is assumed that the other 104 miles of failed pipe has already been replaced during previous renewal projects.

Due to significant project-specific sunk costs (mobilization, contracting, etc.), customer impacts, and the impact of construction activities (traffic, noise, potential repaving, etc.), it is prudent to assess pipe performance and characteristics immediately adjacent to the failed pipe to determine whether extending the project boundaries makes business sense. LWS applies similar logic when determining the extents of a particular replacement project. Therefore, cost effectively addressing the backlog of 48 miles of active Isolated Pipe will require replacing more than 48 miles of pipe.

The average length of LWS condition-based replacement projects since 2008 is 1,683 ft. The average length of currently failed Isolated Pipe is 897 ft. Assuming this ratio continues in the future, this would expand the current backlog by approximately 88 percent. However, replacing this additional pipe should reduce future need. For the purposes of this study, it will be assumed that replacing unfailed pipe will reduce future replacement needs by approximately 75 percent.

5.3 Long Term CIP Investment Scenarios

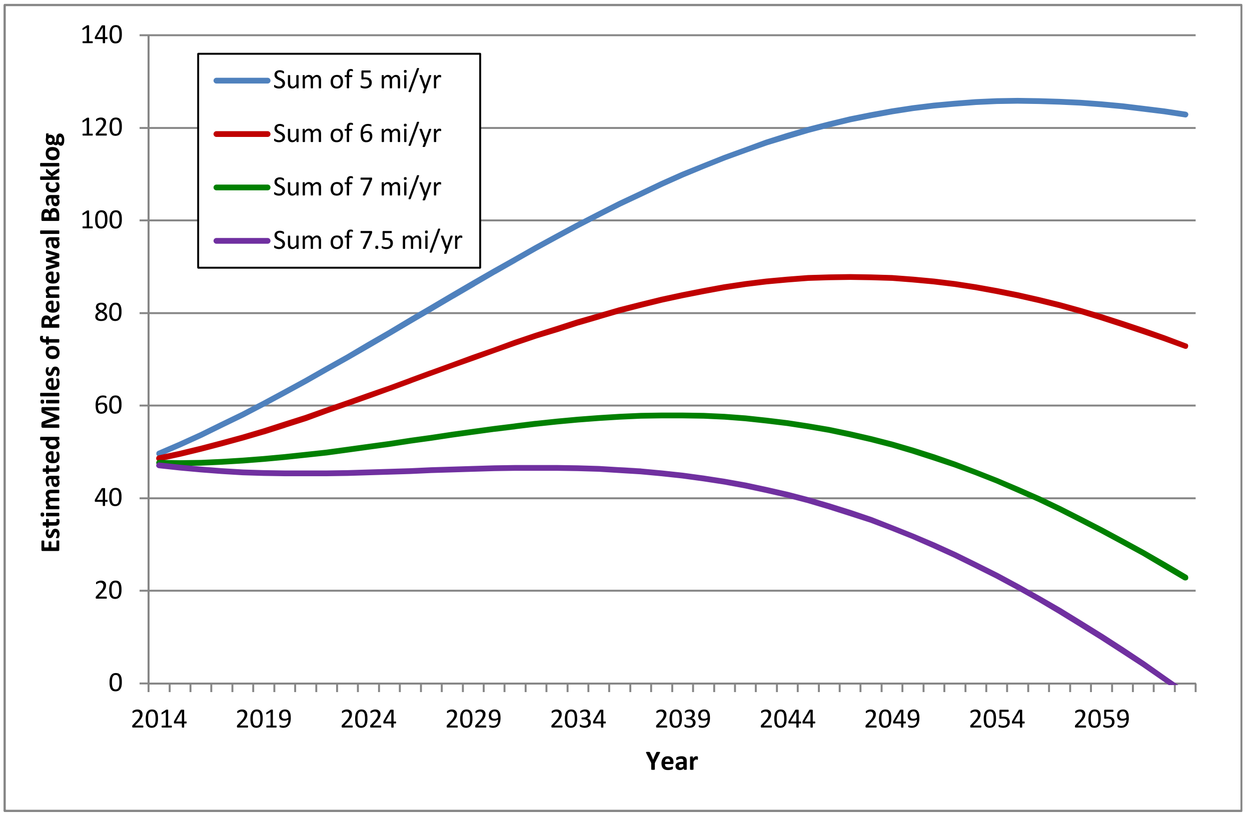
The 2007 Master Plan recommended an annual replacement funding level of $6.92 million. Using the assumption that the replacement projects will cost approximately the same (on a linear foot basis) as the replacement projects that occurred in 2013 (plus inflation), the 2007 Master Plan recommended an annual replacement length of approximately 7.7 miles.

Four 50 year main replacement investment strategies have been developed based on the following scenarios:

- Approximate Current Level of Replacement (5 miles per year)
- Minor Increase in Replacement Level (6 miles per year)
- Maintain Constant Backlog (7 miles per year)
- Eliminate backlog over next 50 years (7.5 miles per year)

Figure 5-1 shows the estimated miles of main replacement backlog over time in each of these scenarios. Like many utilities in the US, LWS has a goal to maintain the current level of service. The model developed estimates that a renewal rate of 7 miles per year will maintain the current renewal backlog.

However, as LWS continues to refine the project identification and prioritization approach, the ratepayer continues to see more return on their investment. Therefore, it may be possible to maintain the current performance at a slightly lower renewal rate. Based on LWS's current performance, desired level of service, cost targets, and risk tolerance; an investment level of approximately 7 miles of pipe replacement per year is recommended. Note, the long term reinvestment level should be periodically re-evaluated to account for changes in system performance, desired level of service, cost targets, and risk tolerance.



Notes:
Current backlog was defined as the number of currently active Isolated Pipes with 3 breaks or more.
Future backlog growth was determined by the Weibull model (less the assumed size of the reinvestment program).

Figure 5-1 Estimated Backlog under 4 Replacement Scenarios

5.4 Summary of Analysis Limitations

When using a model like this to forecast long-term CIP levels, it is important to consider that this model has certain limitations, including:

- This method does not account for the second wave of replacement needs in the out years of the model. That is, a small percentage of the pipes that are replaced in the near term may need to be replaced again within this 50-year projection. This does not significantly impact the overall CIP projection.
- Based on the definition of failure, only 3 percent of assets have failed.
- By count, approximately 52 percent of the Isolated Pipe modeled was built prior to 1984 when readily available break history was first collected (this is anticipated to bias the model to a slightly higher useful life).

- The average age of the system modeled is only 41 years.
- This model does not account for variations in the failure by other secondary characteristics that influence performance such as pipe diameter.
- This model does not account for failures arising from hydraulic deficiencies, water quality deficiencies, or the need to move the pipes due to redevelopments and roadway realignments.
- As more data are collected over time, the accuracy of the model will increase.
- Breaks that are widely separated in time may be random events rather than caused by a common aging process (i.e., corrosion). It may be useful to further investigate how prevalent this is.
- Although protected DI and PVC have been treated as one class of pipe, this is an oversimplification. As more break data is collected, these pipe materials should be analyzed separately.

6.0 Project Prioritization

6.1 Methodology

After analyzing LWS system data, HDR developed a relative risk assessment score. The intent of the relative risk score is to define a consistent, transparent, and defensible approach for prioritizing water main replacement projects. The relative risk score is not intended to replace the need for planning staff to evaluate the extents and/or priorities of particular renewal projects. Rather, it is meant to focus these resources by triaging Isolated Pipe by relative risk. The relative risk model should be updated regularly to account for new data such as break history. As the program continues to mature, it is anticipated that the relative risk methodology will adapt to changing drivers, experiences, and readily available information.

This methodology estimates a relative risk score based on the assigned weights of the following factors:

- LWS Average Assessment Score
- Break Count
- Annual Breaks per 100 miles
- Date of Last Break
- Pipe Diameter
- Pipe Length

- Installation Year

. Figure 6-1 displays the Isolated Pipes in the LWS system with at least 3 breaks. The pipes are assigned a color on a scale that is indicative of respective score .

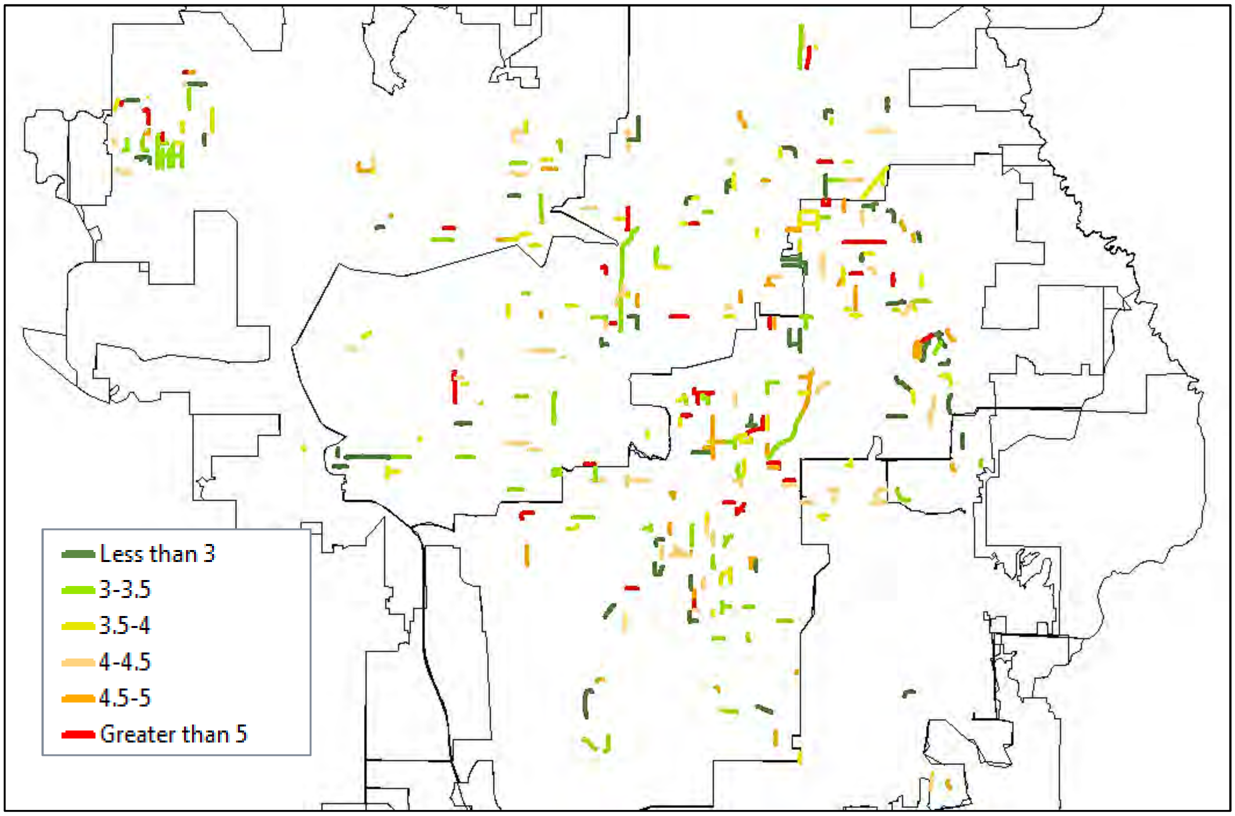


Figure 6-1 Map Displaying Isolated Pipes with a Break Count Greater Than Three

6.1.1 LWS Average Assessment Score

LWS assesses the condition and consequence of failure for a given asset during a repair using a "Distribution System Repair and Condition Report". LWS staff assign a ranking for the Level of Service Consequence (A), Damage Consequence (B), and Water Main Condition and Failure Risk (C). An overall score is calculated for each asset using the following equation:

Equation 3: Average Assessment Score = (A + B) * C

This factor is assumed to require the greatest weight (30 percent) because it captures the consequence of failure of the asset and the condition of the asset through the judgment of field staff.

6.1.2 Break Count

As shown in Figure 3-4 in Section 3.3.1, there is a strong relationship between break count and the duration to the next break within the LWS system. LWS has stated that their goal to maintain an annual break rate of 14 breaks per 100 miles of pipe. In order to accomplish this, it is recommended to give priority to pipes with a higher break count because the duration to the next break will be lower than pipes with lower break counts. A weighting of 20 percent was assigned to this factor.

6.1.3 Annual Breaks per 100 miles

The annual breaks per 100 miles (i.e., Break Rate) are calculated for the Isolated Pipe between the first break and 12/31/2012 (i.e. the last date where break data was used for this analysis). For example, if a 528 foot Isolated Pipe (0.1 miles) first broke in 12/31/2010 and broke again in 2011 and 2012, the break rate would be calculated as:

Break Rate = (3 breaks) * 100 / (3.0 years) / (0.1 miles) = 1,000 annual breaks per 100 miles

This method places additional emphasis on recent breaks and on shorter pipe where more bang for the buck may be realized. A weighting of 20 percent was assigned to this factor.

6.1.4 Date of Last Break

The date of the last break places additional emphasis on Isolated Pipes that have had a recent break. For example, all other factors being equal, an Isolated Pipe that broke last year will be rated higher than a pipe that has lasted several years since its last break. The date of the last break is assigned a weight of 10 percent.

6.1.5 Diameter

The Isolated Pipe diameter is considered in the relative risk score. In general, the score generally reflect an increase in the consequence of failure of a pipe as the diameter increases. That is, generally, a larger diameter pipe has a greater consequence of failure than a smaller pipe. Note, the methodology also prioritizes 4-inch pipe due to general concerns that 4-inch pipe may need to be upsized to meet fire flow standards. Fire flow deficiencies were identified in areas of the system with 4-inch mains and 6-inch non-looped mains in older areas of the City, including the downtown area. When replacing 4-inch and 6-inch mains, improvements should be confirmed with additional hydraulic modeling in each project area to determine upsizing and looping requirements to ensure proper fire flow is made available. Also, improvements should be coordinated with on-going condition and main break evaluations to replace poor condition pipe with new upsized pipe to address both issues at the same time. The diameter is currently assigned a weight of 10 percent.

6.1.6 Material Quality

The analysis has shown that generally, pipes installed between 1941 and 1972 (Thin Walled CI and Unprotected Ductile Iron) are deteriorating at a faster rate than pipes installed between 1884 and 1940 (Vertical Pit Cast Iron) and that pipe installed between 1973 and 2012 (Protected Ductile Iron and PVC) are deteriorating at the slowest rate. This factor accounts for the relative asset class deterioration rates. The material quality is currently assigned a weight of 5 percent.

6.1.7 Length

LWS replacement projects are generally about 1,500-3,000 ft long. Projects where a significant number of breaks are occurring on shorter Isolated Pipe also have the benefit of potentially replacing adjacent pipe that is in need of replacement. Isolated Pipe longer than 3,000 ft is generally more expensive to replace. This factor accounts for these considerations. The length is currently assigned a weight of 5 percent.

7.0 Assessment Alternatives for Transmission Pipelines in Lincoln

Historically, LWS has not experienced a significant number on transmission mains (greater than 24-inch in diameter) nor have there been signs of issues with the condition of these transmission mains. However, several of the transmission mains were installed over 50 years ago and as a result, a systematic process for condition assessment of these mains to determine the need for any rehabilitation or replacement would be beneficial to maintain the integrity of the system. While condition assessment of smaller mains in the system can be cost prohibitive, field investigation and testing of the larger lines and transmission lines can yield benefits in determining the true condition of the pipelines.

Condition assessment methodologies of the larger transmission mains in the LWS were considered. Table 7-1 lists alternatives for such condition assessment. For each pipe, the recommendation is to start with a desktop study (step 1), that looks at available information, the options for assessment, and costs for assessment. From this study, recommendations would be developed for more detailed and more costly steps. In the table below, general progress would be downward from (1) to (2), to (3), but some of the steps can be skipped, depending on the results of the previous steps. Costs for each of these steps can vary significantly depending on the project.

Table 7-1 Condition Assessment Alternatives

Pipeline	Condition Assessment Alternatives
36" cast iron mains • Constructed in the 1920s and 1930s • Operated at pressures up to 100 psi • No cathodic protection and not electrically continuous	(1) Desk-top analysis of drawings, GIS, repair records, soil information. Meeting / interview LWS staff. Site walk to evaluate accessibility, traffic, utility congestion. Analyze data and develop recommendations and cost estimates for field assessments. (2) Field corrosivity survey, including over-the-line measurements of resistivity, cell-to-cell measurements, collection/analysis of soil samples. (3) Leak detection using leak-noise correlators (e.g., Echologics) or in-pipe device (Pure's "SmartBall") (4) Excavation and external direct assessment in locations where corrosion is believed most severe; this would include scanning with Broadband electromagnetic or ultrasonic gauges and pit gauges. (5) In-pipe, end-to-end scanning using remote field electromagnetic tool (PICA's "See Snake"). This would require construction of launching and receiving ports.
48" RCCP Constructed in the 1950s • Operated at pressures up to 150 psi • No cathodic protection and not electrically continuous	(1) Desk-top analysis of drawings, GIS, repair records, soil information. Meeting / interview LWS staff. Site walk to evaluate accessibility, traffic, utility congestion. Analyze data and develop recommendations and cost estimates for field assessments. (2) Field corrosivity survey, including over-the-line measurements of resistivity, cell-to-cell measurements, collection/analysis of soil samples. (3) Leak detection using in-pipe device (Pure's "SmartBall" or "Sahara") (4) Excavation and external direct assessment in locations where corrosion is believed most severe; this would include scanning with Broadband electromagnetic and sounding with hammers. (5) In-pipe manned inspection, including sounding of the pipe to assess delamination locations. This requires dewatering of the pipe. (6) In-pipe, end-to-end scanning using remote field electromagnetic tool (PURE's "Pipe Diver"); the Pipe Diver is very effective for PCCP, but less effective for bar-wrapped pipe. This may require construction of launching and receiving ports or the dewatering of the pipe.

Pipeline	Condition Assessment Alternatives
Welded steel pipe (54-60-inch) • Designed for at least 150 psi (maybe more) • Has cathodic protection and is electrically continuous	(1) Desk-top analysis of cathodic protection (CP) records, drawings, GIS, repair records, soil information. Meeting / interview LWS staff. Analyze data and develop recommendations and cost estimates for field assessments. (2) Field assessment of CP system functionality and effectiveness. (3) Leak detection using leak-noise correlators or in-pipe device. (4) In-pipe, end-to-end scanning using magnetic flux leakage (Pure). [It is very unlikely that we would recommend this for a pipe that is relatively young and under cathodic protection.]

8.0 Available Low-Dig and No-Dig Water Main Replacement Methods

Table 8-1 lists the common water main rehabilitation technologies. Each of these methods is appropriate for the rehabilitation of water mains, depending on the structural condition of the existing pipe, and other considerations. The selection of which system to use generally depends on cost, owner preferences, and other factors. All materials in contact with water should be tested and certified in accordance with ANSI/NSF61 requirements.

Table 8-1 Common Water Main Rehabilitation Methods

Description	Advantages	Limitations
CML, spray-applied, in situ (ANSI/AWWA Standard C602)	• Low cost • Time-tested protection against internal corrosion • Service reconnection not required	• "Non-structural"—not recommended if pipe is structurally deficient • Not recommended where water is soft
Polymer lining, 1 mm thick (epoxy, polyurethane, or polyurea), spray-applied, in-situ (ANSI/AWWA Standard C620)	• Low cost • Time-tested protection against internal corrosion • Service reconnection not required • Rapid set-up of some linings may allow same-day return to service (avoiding bypass system costs)	• "Non-structural"—not recommended if pipe is structurally deficient

Description	Advantages	Limitations
Polymer lining, 3 to 8 mm thick (epoxy, polyurethane, or polyurea), spray-applied, in-situ	• Moderate cost • "Semi-structural"—proven ability to span holes and gaps. • Service reconnection not required • Rapid set-up of some linings may allow same-day return to service (avoiding bypass system costs)	• Not likely to survive fracturing of the pipe[1] • Ability to serve as fully structural system has not been confirmed
Cured-in-place pipe lining, reinforced with fiberglass, polyester or carbon fibers	• Fully or semi-structural • Appears capable of surviving pipe fracture[2] • Robotic service restoration is possible in many cases	• More costly than spray-applied linings • Service reconnections are required, but many can be performed by in-pipe robot • Long-term performance of some products not proven
Tight-fit HDPE slip lining, using roll-down, swage, or deformed methods	• Semi- or fully structural • Capable of surviving pipe fracture • Design criteria and properties are well established	• More costly than spray-applied linings • Service reconnections are required • Limited wall thicknesses available
Pipe bursting replacement	• Fully structural • Some upsizing possible • Design criteria and properties are well established • Compared to tight-fit lining, pipe materials should be more easily procured (less critical sizing requirements and different materials can be used)	• More costly than most other methods, although competitive market exists (not proprietary) • Service reconnections are required • Long-running cracks have occurred with fused PVC, but HDPE is very crack resistant
Cathodic Protection Retrofit	• Can economically extend the lives of water mains • Low-dig methods are available, using vacuum excavation and "keyhole" tools • Can be used in conjunction with in-pipe non-destructive evaluation to target corroded pipe	• Where mains are electrically discontinuous, protection is limited

Notes:

1. Testing will soon be conducted at the Trenchless Technology Center of Louisiana Tech University.

2 Per testing performed at the Trenchless Technology Center of Louisiana Tech University

The rehabilitation techniques listed here are methods that have proven their effectiveness in water main rehabilitation. Many other techniques are promoted, but not all are effective, efficient, or durable. Method selection depends on many site-specific factors, including the structural integrity of the host pipe, the locations and numbers of valves, laterals, and connections, future system plans, and the owner's preferences.

Typically, a pipeline rehabilitation project will concurrently include upgrades or replacements of valves and other appurtenances, such as hydrants, meters, and substandard service laterals, particularly those with lead pipe.

8.1 Case Studies – Utilities Employing Large-Scale Rehabilitation Programs[3]

Several utilities have developed cost-effective programs using these low-dig methods. For example:

- Pipe bursting. WaterOne, the utility that serves several Kansas City suburbs, decided to try pipe bursting for routine water main replacement, using their own construction crews. The utility hoped that pipe bursting would produce cost saving of about 15 percent, by reducing the amount of repaving that is required. In reality, the cost savings exceeded 25 percent because work proceeded more quickly—more footage was accomplished each day. A similar story, but with more remarkable cost savings has been reported by Western Slope Utilities, the utility that serves Breckenridge, Colorado. Western Slope reports 50 percent cost savings.
- CML. Several utilities in North America, Australia, and Europe have routinely employed CML to improve water quality and hydraulic performance, while extending the lives of their water mains. Los Angeles Department of Public Works, Sydney Water in Australia, and many other large cities have in-fact completed the lining of all unlined cast-iron pipe in their systems, and have experienced reductions in break rates as a result. CML is typically performed at less than half the cost of replacement[4].
- Cathodic protection retrofits. Each year, the City of Calgary scans a small portion of its system, using the remote-field electromagnetic method and uses the results to determine which mains are best suited for cathodic protection retrofits. Several criteria are used to select mains for scanning, including the corrosivity of the soil, the history of leaks and breaks, and whether a scanning tool can be readily deployed. Through this program, the number of breaks has been cut in half, paying twice over for the cost of

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[3] Except as otherwise noted, these case studies are from WaterRF Report 4367, "Answers to Challenging Distribution Infrastructure Questions", Ellison, et al., 2013.

[4] Klopfer, Danny J. and Jeff Schramuk, "A sacrificial anode retrofit program for existing cast-iron distribution water mains", *Journal AWWA*, December 2005.

the inspection and retrofits. In Des Moines, Iowa, a pilot program demonstrated that a 20-year life extension was achievable at a cost of less than 10 percent of open-trench replacement .

8.2 Applicability to Lincoln Water System

The performance of distribution mains in the Lincoln system reflects historic differences in the materials that were used and in how the mains were constructed. By breaking the pipes into different asset classes, we can better discern the effects of these differences and forecast future pipe performance.

The major demarcations in the industry were the transitions from horizontal casting, to vertical casting, then to spin casting, followed by factory mortar lining, and the adoption of ductile iron and other modern pipe materials. These transitions did not occur abruptly, with all manufacturers suddenly changing from one manufacturing method to another, and with all utilities suddenly adopting new specifications and standards of construction. Instead, the boundaries between different asset classes are generally fuzzy, as one material gradually replaces another. For LWS, the asset class boundaries have been selected based on what is known for the industry in general and what is seen in the performance data itself.

8.3 Vertical Pit Casting (1884 through 1933)

Due to its very thick walls, pit cast pipe can last a very long time. It simply takes much longer for corrosion pits to penetrate a thick pipe wall, and for corrosion to critically affect its hoop strength and fracture resistance. Furthermore, because rust and other corrosion products act to shield the underlying metal from corrodants, the corrosion rates become slower and slower as time goes on. The combination of thick walls and slowing corrosion rates can result in average life expectancies of 150 years and more. This is reflected in the LWS pipe break data, which show that 60- to 80-year-old pit cast pipe still has low to moderate break rates. 195 miles of pipe in the LWS fall into this category.

Although the performance of this pipe is relatively good, the break rate climbs steadily with each passing year. By cleaning and lining this pipe in place, it should be possible to slow the deterioration (by essentially stopping the interior corrosion) and postpone many repairs (by plugging small holes and spanning over small weaknesses). Such slowing of break rates has been demonstrated in Los Angeles and elsewhere where large-scale lining programs were implemented. With a large scale program, lining can be accomplished at approximately 1/3 the cost of conventional main replacement. Cleaning and lining also significantly reduces water quality risks, particularly the water discoloration, taste and odor complaints, disinfectant depletion, and positive coliform tests associated with unlined cast iron mains.

Alternatives to CML for this class of pipe include polymer lining, reinforced cured in place pipe (CIPP) lining, and pipe bursting replacement. Polymer lining is recommended instead of CML, if the water in the system is unusually soft (<55 mg/l as $CaCO_3$). CIPP and pipe bursting replacement are recommended for pipes where there's a concern about remaining structural integrity, but the cost savings with these methods are less significant than for spray-applied linings.

Candidates for CML would be those pipes where no recorded leaks have occurred. A lack of breaks is often taken as evidence that either the soils are not overly aggressive, or that corrosion products have partially passivated the pipe surface. While these assumptions are not perfect, very few break repairs will be needed for several decades following most cement lining rehabilitations.

The life expectancy of these older pit cast pipes can also be effectively extended through anode attachment. As mentioned earlier, Calgary selects pipe for anode attachment by using in-pipe scanning tools. If the pipe exhibits significant corrosion but only moderate loss of integrity, it is considered a prime candidate for anode attachment. Many other utilities routinely attach anodes to pipes during the course of making main repairs. Because pit cast pipe has lead-caulked joints, pipe-to-pipe electrical continuity often exists, allowing an anode to provide protection to several nearby pipe segments.

8.4 Progressively Thinner Wall Spun Cast Iron (1934 to 1947)

This class of pipe has some of the highest rates of failure of pipe in the Lincoln system. Curiously, the younger pipe in this class (e.g., 45- to 55-year-old pipe) has higher rates of failures than the older pipe (e.g., 60- to 70-year-old pipe). This reverse trend may be due to the fact that pipe in this era was made progressively thinner and thinner, as manufacturing methods got better and better. The consequence was that pipe became successively more vulnerable to deterioration. Approximately 45 miles of pipe in the Lincoln system fall into this asset class.

Like the earlier pit cast pipe, this pipe is believed to be installed without CML, and is thus more vulnerable to interior corrosion. CML became commonly available around 1940 and became prevalent after WWII. Prior to this, asphaltic or bituminous linings were sometimes applied, but provided temporary protection at best.

As with vertical cast pipe, this early spun-cast pipe may be suitable for CML, if no historic leaks have been recorded. CML will improve system hydraulics and minimize water quality risks, while extending the life expectancy of the main. Where breaks and leaks have occurred, reinforced CIPP and pipe bursting replacement are more appropriate.

8.5 Unprotected Ductile Iron and Thin Walled Cast Iron (1948 to 1972)

While CI continued to become progressively thinner, by the late 1940s CML became common. This improved pipe performance significantly by largely eliminating the tuberculation that leads to water quality problems and hydraulic restrictions. Presumably the cement mortar lining also extended pipe lives by largely eliminating interior corrosion, but any improvement in break performance was largely negated by the thinning of the pipe walls. The 309 miles of pipe in the LWS system fall into this category.

Because these pipes are already cement mortar lined, their performance would not be significantly improved with new, non-structural linings. Therefore reinforced CIPP lining and pipe bursting replacement are the only viable alternatives to open-trench replacement, while anode attachment may also be an option worth considering for life extension.

8.6 Modern Era Pipes including Protected Ductile Iron and PVC (1973 to Current)

Improved performance is expected from DIP for two reasons. First, the improved ductility of the material reduced the consequences of iron pipe failure, greatly reducing risks. Second, through the use of PE sheet encasement, the notion of corrosion protection for iron pipe was broadly introduced. Although unprotected iron pipes are still installed in many locations, the PE “baggie” is now common in most utilities. At a minimum, PE encasement is needed for good long-term performance in all but the most benign environments.[5] Where DIP is properly installed with PE encasement, long-term break rates are expected to be relatively modest except in the most aggressive environments.

PE sheet encasement is no guarantee against leaks or breaks. In fact, imperfections in the encasement can rapidly lead to through-wall pits simply because corrosion may concentrate where the PE sheeting is torn. This is particularly true where copper lateral pipes connect directly to the iron pipe. Galvanic action between the iron and copper will concentrate in the small area where the iron contacts the soil. However, despite a through-wall pit, the rest of the pipe will be largely undamaged, with a long remaining life expectancy. If repairs are well made and the PE sheet is fully restored, decades may pass before another break occurs on the same main (assuming that the PE defect was an anomaly). Life expectancy can be further enhanced if a sacrificial anode is also installed at the leak repair location.

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[5] Contrary to some beliefs, the corrosion of ductile and cast iron occurs at roughly equal rates through roughly equal mechanisms, and the black asphaltic coating applied to DIP in the factory provides little protection.

Water mains constructed with PVC are likewise expected to provide low break rates well into the future. Based on research by the Water Research Foundation,[6] a break rate of 10 per 100 miles per year may be expected for 100-year-old PVC pipe. This break rate can be further substantially reduced by:

- Protecting the pipe from scratches and gouges during installation (and not over-stabbing the bells)
- “Over designing” the pipe; for example, using Class 200 PVC where Class 150 is required reduces future breaks by 80 percent
- Using proper tools for tapping and avoiding taps on the outside of curved mains

Notwithstanding its good long-term performance, instances of premature breaks may warrant detailed forensic investigations. One manufacturer was recently found liable in a class action lawsuit by dozens of utilities for the use of substandard materials.

9.0 Recommendations

9.1 Data Collection Improvements

After running the initial spatial join of active pipe network and breaks, there were 196 breaks that had negative ages, meaning that the pipe broke before it was installed. This likely indicates that either the break was joined to the incorrect pipe, or there was a replacement project that was not documented. In either case, it is recommended to associate a break to a pipe shortly after the break occurs to better facilitate data management and analysis as pipe is replaced or added to the system. This will help to avoid issues associated with spatial joins and data processing.

A number of pipes were found to have the date of replacement for the abandoned pipe instead of the original installation date. It is recommended that the original installation date should remain unchanged when abandoning or removing a pipe from GIS.

It is recommended that LWS consider the condition assessment score with the consequence of failure to appropriately prioritize investments. Where the consequences of failure are high, direct assessment methods should be used periodically to determine pipeline condition and take preventive actions as appropriate.

[6] WaterRF Report 2879, “Long-Term Performance Prediction for PVC Pipes”, Burn, et al., 2005.

9.2 Recommendations to Existing Programs

LWS currently has a valve and hydrant inspection program which is detailed in Section 2.3. Hydrant valves are located and documented, but are not exercised since they do not affect customer service or fire protection provided. It is recommended to exercise the hydrant valves during this inspection.

AWWA M44 Distribution Valves: Selection, Installation, Field Testing, and Maintenance recommends performing valve inspections annually if possible with 16-inch and larger valves being inspected more frequently. With the large number of valves that LWS maintains, it is not practical to attempt to inspect all of the valves annually. Inspecting the large diameter valves more frequently or annually as recommended in the 2007 Master Plan should be reconsidered. The impact of a larger diameter valve not functioning when needed could include the following:

- A larger service area would be affected by a shut down on valves further away from the maintenance area. This area would be significantly larger than a similar inoperable valve on a 6-inch or 8-inch main.
- In the instance of a main break on a large main, more water would be lost while attempting to shut-off flow to the break. The risk of contamination would affect a larger part of the system.

9.3 Material Recommendations for New Installations

LWS is presently allowing both PVC and DIP for new water main installations. If installed with care, both PVC and DIP should provide long lives with low failure rates. PVC is generally less costly to install, but the cost difference is not great.

The performance of PVC is relatively predictable. PVC tends to fail by slow-crack growth, starting from defects and gouges in the material. PVC failures are brittle, with cracks typically several ft long, but occasionally 20-ft long (bell to spigot). Because PVC fails by cracking, its use should be limited to smaller diameter pipes (e.g., 12-inches and smaller) in areas where a large break would not be catastrophic. The WaterRF[7] study predicts approximately 10 breaks per 100 miles per year when a pipe is 100 years old.

[7] WaterRF Report 2879, “Long-Term Performance Prediction for PVC Pipes”, Burn, et al., 2005.

The PVC break rate can be greatly reduced by “overdesigning” the pipe—using Class 200 when Class 150 is required. This will result in only about 2 breaks per 100 miles per year, at an age of 100 years.

PVC should not crack during tapping if the tapping is performed correctly. PVC will crack during tapping if the wrong bits are used, the tap occurs on the outside edge of a pipe bend, or the PVC material is bad.

The life-cycle performance of DIP is less predictable, because it depends on corrosivity of the environment and how well the pipe is protected from corrosion. In some cases, pipes will last hundreds of years. In other cases, the pipe needs replacement after 20 years due to rapid corrosion. A typical DIP failure is a rust-hole leak, rather than a crack. DIP should have PE wrapping at a minimum, for protection against external corrosion, and should be electrically isolated from copper services. It is recommended to have LWS’s current specifications reviewed and updated as appropriate.

9.4 Project Prioritization and Risk Assessment Protocol

The relative risk model should be updated regularly to account for new data such as break. As the program continues to mature, it is anticipated that the relative risk methodology will adapt to changing drivers, experiences, and readily available information. The current analysis extends through the end of 2012. It is recommended to annually update the project prioritization spreadsheet with the breaks for the past year. This will allow LWS to re-prioritize water main replacements for upcoming years using the most current data available. It is also recommended that LWS staff review the current weighting of the criteria listed in Sections 6.1.1 through 6.1.7.

9.5 Future Inspection Programs & Planning Strategies

9.5.1 Conclusions Regarding Pipe Renewal in the LWS

Through its current main renewal program, the City has kept its system-wide break rate below 20, which would be considered moderate and sustainable. This has been accomplished by replacing mains with significant numbers of breaks using conventional open-trench methods.

As indicated in the previous section, HDR recommends a slight increase the length of water main replaced annually as a part of the LWS main replacement program. Through the adoption of an alternative renewal strategy, Lincoln should achieve this, without a significant increase in cost. This would ultimately reduce the long-term break rates and help to better manage risks. Possible components of such a strategy are:

- The adoption of pipe bursting or reinforced CIPP lining as a standard method for main replacement, in lieu of most open-trench replacement.[8]
- The routine attachment of sacrificial anodes during break repairs.
- The possible implementation of a lining program for unlined cast-iron pipe, if water quality concerns warrant it. This program would target pipes without a history of breaks.

Field condition assessment and focused rehabilitation for transmission pipelines where the consequence of failure warrant such special attention.

9.6 Sustainable Level of Investment

Based on the results presented in Section 5.3, an annual replacement of 7 miles is recommended for LWS. To support this level of replacement approximately, $6.3 million (in 2014 dollars) is required for a sustainable level of investment for the water main replacement program. This estimated cost is based on the assumption that the replacement projects will cost approximately the same (on a linear foot basis) as the replacement projects that occurred in 2013 (plus inflation).

In general, there is a balance between the size of the water main replacement program and the number of field crews needed to repair main breaks. A larger water main replacement program could require more engineering resources to design or manage the design of replacement projects and more operations staff during the construction phase of the project.

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[8] It is generally preferred that one method be selected. This avoids the need for additional training and material storage.

Lincoln Water System Facilities Master Plan

Chapter 7 - Asset Management

May 2014

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Abbreviations and Acronyms

AHU	Air Handling Unit
BSA	Business System Analyst
CMMS	Computerized Maintenance Management System
City	City of Lincoln
GIS	Geographic Information System
HDR	HDR Engineering, Inc.
LWS	Lincoln Water System
VFD	Variable Frequency Drive

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1.0 Introduction

The City of Lincoln (City) performed a previous asset management needs assessment with CH2M Hill in 2009. Through this assessment, a comprehensive asset management program was identified as both a gap and a priority in continuing high performance operations and organizational sustainability at the Lincoln Water System (LWS). The purpose of Chapter 7 – Asset Management Program is to outline a practical and tangible approach to guide LWS to comprehensive asset management.

2.0 Assessment Methodology

The methodology used as a part of this Master Plan to conduct the asset management evaluation for LWS compared currently used business processes against industry best practices in the context of the Asset Management Framework, as depicted in Figure 2-1. The framework items identified in the blue boxes in Figure 2-1 were the focus of this study:

- Asset knowledge is generally obtained and maintained in the computerized maintenance management system (CMMS).
- People and processes drive the asset management program.

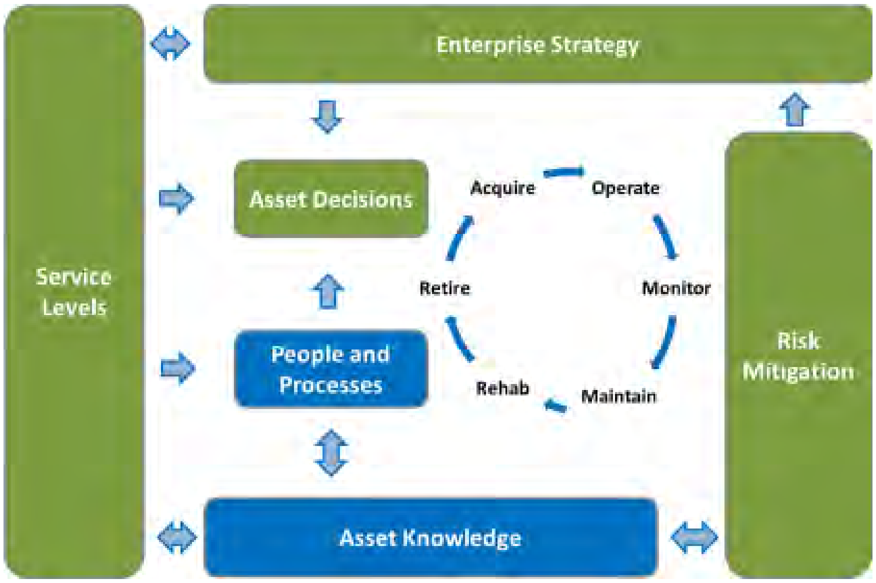

Figure 2-1 Asset Management Framework

For this evaluation, data was acquired and reviewed from a Geographic Information System (GIS) and the LWS CMMS, Hansen 8.2.3. A short overview of systems and business processes was discussed with LWS leadership and the CMMS administrator and GIS analyst who support LWS.

3.0 Current Asset Management Program

3.1 Asset Knowledge

In general, LWS has a very up-to-date IT infrastructure. Most software is on the latest release and there are plans to acquire a new virtual server. It is apparent that staff value IT systems.

3.1.1 Hansen CMMS

LWS uses Hansen 8.2.3 CMMS and recently moved from Oracle to SQL Server 2008 r2. The database is stored on the City servers at an offsite location. Although LWS staff has administrative permissions on the database, they do not have the tools to administer the system. Ultimately, the administration and responsibility of the data lies within other divisions of the City.

3.1.2 GIS

The GIS system appears to be a robust data model for horizontal assets. However, a single authoritative asset inventory database has not been established for LWS and it is recommended that GIS serve, at least in part, as the authoritative asset inventory. Data from GIS has not historically been synced with the CMMS, which can cause discrepancies in data used for analysis. GIS data is stored in a file geodatabase on a file server, not in an enterprise database that can be accessed City-wide.

A key factor in streamlining the process of syncing the GIS and Hansen is the labeling of attributes, or primary keys, in GIS. The existing primary keys include spaces and null values, which can create issues in Hansen. To facilitate syncing of these systems, strong primary keys should be established as part of a data-model makeover prior to syncing.

Spatial services do not drive spatial business systems or business decisions; this is a level of maturity that would be desirable to optimize interoperability of disparate IT Systems. As seen in Figure 3-1 in the Simplified GIS Maturity Model, it is desirable for spatial services to inform business applications in a real time, integrated manner. In the later two levels of the maturity model, Integrated Applications and Net-Centric, GIS is working with the business and asset management systems seamlessly to answer spatial questions. An example of this would be a master address web service that systems could query for address verification.

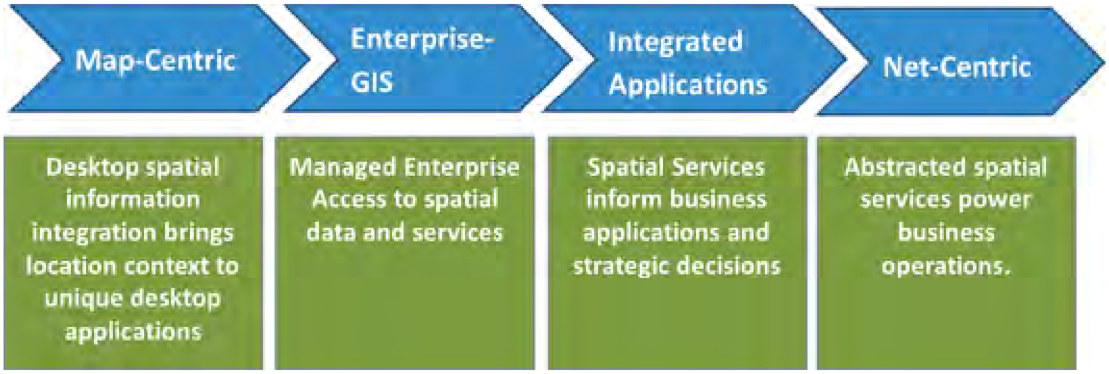

Figure 3-1 Simplified GIS Maturity Model

3.1.3 Integrations

City-wide, several CMMS programs are used as part of the day-to-day asset management programs, including MP2, Cartography and Hansen. A plan for an enterprise strategy and integration of IT asset management systems City-wide could begin to be addressed with the integration of the CMMS with GIS. A workflow for data should be established as it is essential to have data flow “down-stream” as shown in Figure 3-2 and match in all systems. A defined workflow helps establish authoritative data sources while keeping data accurate and timely.

Figure 3-2 GIS Data Flow

As defined in Figure 3-2, asset data should first be obtained from record drawings or plans and input into GIS. GIS would then be hooked or synced with the CMMS. As a part of normal field operations, the data could be confirmed and updated as necessary into GIS.

3.2 People Overview

LWS has a very experienced staff with much of the maintenance activities being conducted based on well developed schedules and system knowledge. While this knowledge and expertise is critical to the overall operation of the system, using an Enterprise Architecture as pictured in Figure 3-3 with the proper IT systems and business processes will facilitate information transfer and will significantly reduce the potential of losing system knowledge as a result of staff changes.

Figure 3-3 Enterprise Architecture Overview

Currently, LWS lacks a dedicated asset management program manager. The current program manager is also the IT systems administrator and the small computers technician. The GIS Analyst also serves in multiple roles.

A separate position is needed to focus on the analysis of data in the asset management systems. Ideally, this staff member will understand the business of water and have the ability to query data in various systems. This is particularly true with the Water Supply and Production section, where the current CMMS contributes little to asset knowledge or business decisions because the data is not in a format that facilitates use and analysis.

Having staff or personnel in the right position and guiding best practices will be critical in moving the program forward. Staffing should be optimized in order to use CMMS in an enterprise manner City-wide. Existing staff could be used to remedy most of these gaps if an asset planning group is organized under a strong leader.

LWS staff has justifiably prided themselves in the commitment to provide outstanding service to customers with exceptional reliability and regulatory compliance. LWS has also received numerous awards for water quality. These accomplishments could be further highlighted using a transparent enterprise CMMS with reports publishing how the organization exceeds levels of service.

3.3 Business Processes Overview

LWS is in various stages of maturity in regards to a City-wide asset management program. The Water Distribution section has furthered the implementation of Hansen and GIS throughout the distribution system and readily uses these systems to prioritize system needs and work order generation. Syncing of GIS and Hansen would further the implementation of an asset management program and provide a single, reliable source of information.

For the Water Supply and Production section, the use of Hansen has not advanced to the same level and generally it is not used for inspections. Service request are intermittently captured and do not relate to work orders, and the asset inventory does not match in IT systems.

4.0 Asset Management Program Recommendations

The focus of the recommendations developed for this chapter of the Master Plan is to optimize the use of Hansen and other asset management process and tools for LWS. Further enhancements to the Enterprise Architecture for all of the Department of Public Works may result through coordination of the asset management processes and tools across multiple divisions within the Department.

4.1 Sync GIS with Hansen

The simplest and most important change that can be made to the Hansen system is to develop a 360 degree workflow for syncing authoritative asset data (GIS) with Hansen as presented in Figure 4-1.

Figure 4-1 GIS/Hansen Workflow

The records in GIS should have attributes that serve as meta-data for the asset; such as:

- Source of the data - plan, as-built, survey, field verified or aerial image (remote sensing)

- Status of asset - planned, built, replaced or retired (abandoned)

Systems should be synced periodically, which is often completed as a “batch job” after work hours to aid in the availability of both the Hansen and GIS. It is not uncommon for systems to be synced nightly. However, weekly syncs are often acceptable timeframes in most organizations.

Before creating a workflow for syncing GIS with Hansen, it is highly recommended that a data-model be developed and documented; it is much harder to make changes to the GIS and/or Hansen after the system has been in use. The data model that is developed needs to consider the relationships of all systems. Primary keys should be developed considering that null primary keys are unacceptable in Hansen. Since GIS is often the most configurable system, it is recommended that it be used as the hub of most integrations, which is referred to as a GIS-Centric integration.

4.2 Develop an Asset Hierarchy

An asset hierarchy is an essential building block in asset management. A hierarchy adds structure to a systems asset registry allowing costs and conditions to roll up to asset groups. There are no hard-fast rules on what a hierarchy should look like. A general depiction of asset hierarchy is presented in Figure 4-2. Development of an asset hierarchy should consider the level of detail the City would like to report on which should be consistent with the smallest “maintenance managed” asset or the level at which financial decisions are made.

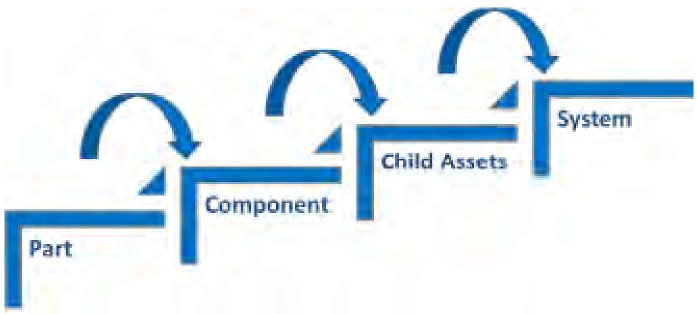

Figure 4-2 Asset Hierarchy

4.3 Define Initial Business Processes

It is essential to develop utility-wide business practices. Work orders and service requests should be defined and used in a uniform, enterprise-wide fashion. It is a recommended best practice to establish a workflow where service requests can be aggregated into “buckets” of work orders; work orders will then be related to assets. This allows for more robust reporting and analytics. It may not be important to immediately define all business processes because as the system matures, the business processes will also mature and change. Figure 4-3 presents a recommended workflow.

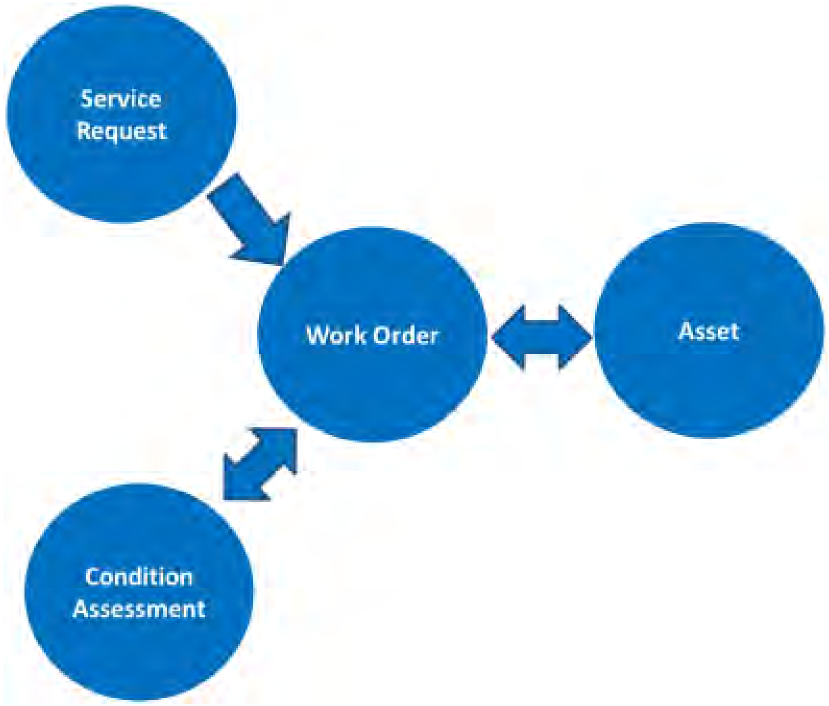

Figure 4-3 Work Flow Diagram

4.3.1 Service Requests

Service requests are customer facing and should be used to track requests by customers. Tracking service requests is useful for reporting on trends, etc. Service requests should be turned into work orders by staff who understand how a particular request would relate to specific work that they perform.

4.3.2 Work Orders

Work orders can be used to aggregate services requests or can be used to assign and track work performed internally. The work order can act as the hub of all transactions. Equipment, labor and materials should be tracked in a work order. In most cases, work performed on a work order should be related to assets.

4.3.3 Condition Assessments

Through recording of condition assessments or inspections in CMMS, work orders can be generated and tracked by asset. Condition assessments will be further discussed in Section 7.0 of this Chapter.

4.4 Centralized Asset Management Team

Ideally, an asset management team would be formed to manage both the CMMS and GIS databases. A business systems analyst (BSA) would lead the asset management team. In general, the business systems analysts would act as the asset management program manager and interface with the City IT group. This will allow the City to build a true single source of data and eliminate redundant and/or out-of-sync data systems. The BSA would lead a team with the following skill sets:

- CMMS Systems Administration
- CMMS Data Quality
- GIS Data Quality and Entry
- CMMS Reporting and Analysis
- Facilitate training programs

4.5 Establish a Training Program

Training is an important aspect of an asset management program specifically as it relates to established business processes and the use of the CMMS. Trained staff will continue to advance the asset management program by instilling best practices. A robust training program eases the anxiety about using software and or new technology. Training is the greatest aid in change management.

A robust training program also expands breadth-of-use of the CMMS. It has been proven that mobile technologies have a very good return on investment. Training field staff to use the CMMS in a mobile environment will increase data quality and make work more efficient.

5.0 Asset Management Action Plan

To assist in the implementation of the asset management recommendations in this chapter, a detailed action plan was developed as presented in Table 5.1. The action plan categorizes each activity by priority and timeframe to further assist with implementation.

Table 5-1 Asset Management Action Plan

Category	Opportunity	Time-Frame1	Priority
People and Processes	Establish a project leader	Immediate	High
People and Processes	Define Initial Business Processes	Immediate	High
Asset Knowledge	Determine assets you plan to track	Immediate	High
Asset Knowledge	Establish a GIS / Hansen data-model	Immediate	High
Asset Knowledge	Develop asset hierarchy	Immediate	High
People and Processes	Sync GIS with Hansen	Immediate	High
People and Processes	Establish a training program for users	Immediate	High
People and Processes	360 degree workflow to sync GIS data with Hansen	Short	High
Risk Mitigation	Outline asset criticality	Short	High
People and Processes	Standardize work codes	Short	Med
Levels of Service	Define Levels of Service	Short	Med

Category	Opportunity	Time-Frame1	Priority
Levels of Service	Track data towards measures	Short	Med
People and Processes	Continue to train users	Short	High
People and Processes	Re-organize staff as a centralized work group	Mid	Med
Asset Decisions	Develop project prioritization criteria	Mid	High
Asset Decisions	Develop preventative maintenance plans	Mid	High
Asset Decisions	Manage risk v. levels of service	Mid	High
Enterprise Strategy	Work towards a paperless organization	Mid	Low
Enterprise Strategy	Implement Mobile technology	Mid	Med
Enterprise Strategy	Train a broader group of users	Mid	Low
Levels of Service	Use KPIs to measure performance against historic values	Long	Med
Enterprise Strategy	Develop Master Plans are developed using real world data	Long	Med
Enterprise Strategy	Maintenance backlogs are prioritized based primarily on the criticality and condition of the underlying assets	Long	Med
People and Processes	Manage Capital Projects	Long	Low
Asset Knowledge	Rate studies match historic trends	Long	Med
People and Processes	Optimize business processes	Long	Low

Note:
1 The implementation time-frames are defined are follows:
Immediate – Next 6 months
Short – 6-18 months
Mid -2-3 years
Long - >3 years

6.0 Facility Condition Assessment Framework

Determining the condition of assets is a critical component of an overall asset management program. A structured condition assessment program enables a utility to collect detailed data regarding the facilities and use that information to optimize maintenance strategies. Condition assessment requires an understanding of:

- Intended function(s) of the asset
- Failure modes
- Required reliability
- Current performance

The focus of the condition assessment component of the Master Plan was to define a framework for condition assessments for the LWS facilities. This framework includes the establishment of recommended protocols so that LWS staff could conduct assessments as scheduled and/or needed system-wide.

6.1 Overview of Condition Assessment Protocol

Condition assessment for facilities requires a comprehensive review by a multi-disciplined team to define the needed improvements, prioritize the improvements, estimate the rehabilitation costs and organize the projects into a financially implementable capital improvement program. The benefits of a condition assessment program include:

- Extending the life of your facilities
- Reducing the incidence of failures
- Saving money over the life of an asset through targeted and preventative maintenance as opposed to reactive repairs
- Protecting public health and safety

6.1.1 Condition Assessment Approach

Condition assessments should be conducted where the risks of failure and/or the consequence of failure of an asset are high. The planning and organizational efforts are keys to the completion of a condition assessment program. The overall assessment approach is summarized in Figure 6-1

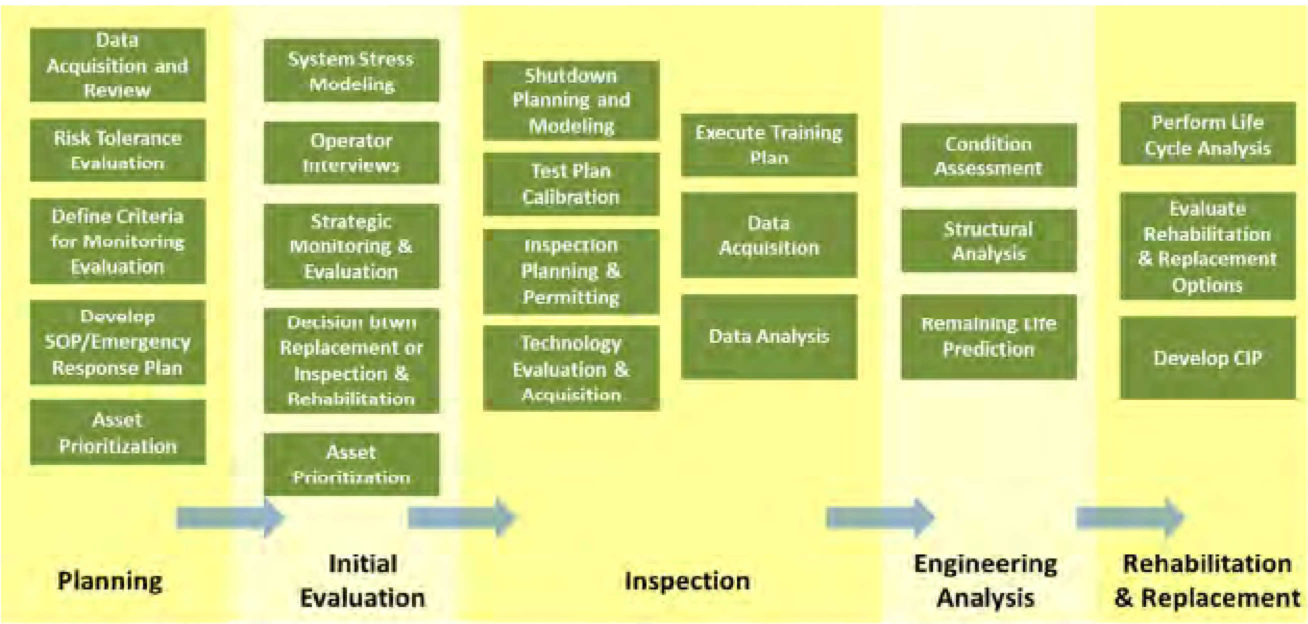

Figure 6-1 Condition Assessment Process

6.1.2 Team and Levels of Inspection

The best approach for condition assessment includes an experienced and trained team working interactively with management, operations and maintenance staff. The condition assessment team should be assembled to match the type of facility being assessed and the level of assessment being conducted. With proper training, facility operations and maintenance staff are the first line of defense in monitoring asset condition, as they regularly use and maintain the facilities and equipment throughout the system. In addition, a multi-disciplined team approach allows all aspects of the facility to be adequately assessed. The selected team members will complete a comprehensive evaluation of existing conditions, age, performance, code issues and spare parts.

The level of detail of a condition assessment can vary depending on the need and condition of the asset. Figure 6-2 presents the various levels of condition assessment.

Figure 6-2 Levels of Condition Assessment

The planning phase of a condition assessment program identifies the level of inspection anticipated. The typical effort includes a Level 2 Observation which may be followed up by more detailed performance testing or even forensic evaluations or destructive testing. The goal is obtain the minimum level of information to develop a sound, implementable improvement plan that can be implemented within the financial constraints of the utility.

6.1.3 Condition Assessment Rating

As a part of a Level 2 Observation, the assets are rated based on visual inspection by the condition assessment team. Depending on the facility, the observation will include an assessment of the architectural, structural, mechanical, electrical, process and instrumentation and controls systems.

To assist in the visual assessment process and to facilitate integration of the condition assessment into the overall asset management program, a rating of the asset based on the condition, capacity and reliability is recommended.

6.1.3.1 Condition

For the evaluation of the condition of an asset, there are five quantitative ratings for the evaluation of each of the critical components of the asset. The five quantitative ratings are as follows:

1. New or Excellent
2. Minor Defects Only
3. Moderate Deterioration
4. Significant Deterioration
5. Virtually Unserviceable

U. Unknown.

The condition rating is accompanied by a percentage of remaining useful life and a maintenance bench mark.

6.1.3.2 Capacity

The capacity evaluation includes consideration of the performance of an asset based upon a review of operating records and discussion with operations and maintenance staff. The capacity of an asset is then rated based on the following five quantitative ratings:

1. Exceeds Rated Capacity
2. Meets Rated Capacity
3. Minor Capacity and /or Performance Issues
4. Significant Capacity Deficient
5. Out of Service

If the capacity rating is 3 or higher, the next level of inspection as presented if Figure 6-2 should be considered for the asset.

6.1.3.3 Reliability

The evaluation of the reliability of the system includes assessing how often maintenance of the asset component is required or if failure is anticipated. To assess the reliability of an asset, discussions with a knowledgeable field technician or the end user are conducted along with a review of maintenance records, if available. There are five reliability ratings:

1. Failure Not Anticipated
2. Random Breakdown
3. Occasional Breakdown
4. Periodic Breakdown
5. Continuous Breakdown

If the reliability rating is determined to be 4 or higher, a higher level inspection should be considered.

To assist in the condition assessment process, HDR Engineering, Inc. has created forms for each discipline (architectural, structural, mechanical, electrical, process and instrumentation and controls). These forms provide a standard form for evaluation and act as a guide for the key considerations for the Level 2 Observation. Examples of these forms will be presented along with the results of the example condition assessment conducted for LWS. The forms and the example condition assessment are discussed in Section 7.2 below.

7.0 Recommendations

A robust asset management system will provide LWS the information and tools necessary to make critical decisions for the system. These decisions include maintenance scheduling and proactive prioritization of capital renewal and replacement projects. LWS has made progress in the implementation of an asset management program with the further implementation and population of GIS and Hansen.

To further advance this system, LWS should develop defined and consistent business processes through out all sections within the division. This includes consistent use of GIS and CMMS, establishment of an asset management hierarchy, and routine syncing of GIS and the CMMS. In addition, an asset management project leader should be identified to facilitate the implementation of the program.

Another critical element of a robust asset management system is the implementation of a condition assessment process. This process will allow LWS to further extend the useful life of assets, reduce the potential of failure, and identify those assets that have the highest potential and consequence of failure in the system.

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Lincoln Water System Facilities Master Plan

Chapter 8 - Financial Assessment

May 2014

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Abbreviations and Acronyms

AWWA	American Water Works Association
CIP	Capital Improvements Program
City	City of Lincoln
DSC	Debt Service Coverage [Ratio]
HDR	HDR Engineering, Inc.
LWS	Lincoln Water System
Master Plan	2013 Facilities Master Plan
MHI	Median Household Income

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1.0 Introduction

1.1 Background

The effective implementation of the 2013 Facilities Master Plan (Master Plan) is ultimately dependent on developing an overall financial plan that can be supported by the utility's revenue sources, while still meeting any relevant State and local regulatory requirements, and providing the flexibility to deal with unforeseen changes. This chapter presents a financial assessment that reviews the revenues and expenses for Lincoln Water System (LWS). The capital costs contained within the financial assessment are based on the Capital Improvement Program (CIP) developed within this Master Plan.

The financial assessment considers both the annual operating costs and capital needs of the water system. The financial assessment determines the financial feasibility of the Master Plan and what adjustments may be needed to the current water rates and revenues to adequately support the Master Plan. At the same time, the financial assessment considers other financial planning criteria such as debt service coverage (DSC) covenants and maintenance of adequate reserve levels.

This study has explored two time periods: a 10-year projection and a 30-year projection. The Lincoln Water System (LWS) has historically conducted master planning efforts on five-year intervals; a comprehensive master planning effort every ten years and updates to address system growth and distribution system needs every five years. The 10-year projection is more critical for immediate financial planning purposes, but developing a financial forecast for a 30-year time period takes a longer range vision of the needed improvements and costs to the system. In addition to the 10 and 30-year financial assessments developed as a part of this study, the financial assessment (analysis) also explored the financial feasibility of the Missouri River water source project that carries an additional and significant financial burden with it.

1.2 Purpose of the Financial Assessment/Review

The objective of this chapter is to provide a very high-level review of the financial feasibility of the Master Plan. This financial assessment is not a comprehensive rate study, and it is not intended to be used for rate setting purposes. This high level analysis should not take the place of the City's annual review and adjustment of water rates. The City has historically reviewed their water rates on an annual basis to determine the appropriate level of adjustments for the rate setting period.

As noted above, this analysis and financial assessment is used to determine the financial feasibility (i.e. the utility's financial capability) in relation to the Master Plan's capital projects. The financial assessment can provide a "road map" for the future to enable a smooth transition, if needed, to fund the capital projects. Finally, this financial assessment can also help identify potential issues related to utility or customer affordability.

2.0 Overview of the City's Current Water Rates

2.1 System Overview

In 2011, the City conducted a comprehensive water rate study for the LWS. This study reviewed both the adequacy and equity of the LWS's water rates. The study also reviewed the need to restructure the water rates to create greater revenue stability, while continuing to provide a strong conservation price signal to consumers. The development of this comprehensive rate study is an industry best practice and utilized generally accepted financial planning and rate setting principles as endorsed by the American Water Works Association (AWWA). In addition to the comprehensive rate study conducted in 2011, the LWS also reviewed the issue of water shortage/drought rates and adopted a set of water management/drought rates in the spring of 2013. Fortunately, the City and LWS did not need to declare a water shortage/drought in 2013 and implement these temporary water management/drought rates. They do remain in place for future use should a drought occur.

The City currently charges its customers a monthly water service charge, based on meter size, and a volumetric consumption charge. In general, the service charge is intended to cover the billing, accounting, and meter reading costs, along with a portion of the maintenance costs associated with the distribution system. Shown below in Table 2-1 is an overview of the current monthly water service charges. These monthly water service charges vary by meter size.

Table 2-1 Monthly Water Service Charges

Meter Size	Rate[1]
5/8"	$3.80
5/8" x3/4"	$5.75
3/4"	$5.75
1"	$9.60
2"	$19.15
3"	$30.65
4"	$57.50
5"	$95.85
6"	$191.65
8"	$306.65
10"	$440.85

Note:
1. Rates apply to residential and non residential customers

In contrast to the monthly water service charges, the consumption charge varies by customer class of service (e.g. residential, non-residential, etc.). The consumption charge for residential customers is based on the amount of water used within a three tiered, increasing block rate structure. This means that the more water a customer uses, the more expensive the per unit rate will become in regards to the three consumption blocks. This rate structure is intended to encourage efficient water use, particularly as it relates to more discretionary outdoor use (e.g. lawn watering). The current consumption charges for residential customers are shown below in Table 2-2.

Table 2-2 Residential Monthly Consumption Charges ($/CCF)

Price Blocks[1]	Rate
1 - 8 Units	$1.344/unit
Next 15 Units	$1.911/unit
All Additional Units	$2.961/unit

Note:
1. 1 Unit = 100 cubic feet or 748 gallons

For non-residential (i.e. commercial) customers, the LWS has a different rate structure to better reflect the differing and distinct usage patterns of this particular group of customers. The non-residential consumption charges are shown below in Table 2-3.

Table 2-3 Non-Residential Monthly Consumption Charges ($/CCF)

Price Blocks[1]	Rate
1 - 80 Units	$1.344/unit
All Additional Units	$1.911/unit

Note:
1. 1 Unit = 100 cubic feet or 748 gallons

It should be noted that the LWS has a third consumption rate for their largest industrial customers, or their high users. A "high user" is any non-residential customer that uses more than 12 million cubic feet of water the previous calendar year[1]. These customers are billed according to the high user schedule. For purposes of this study, a discussion of that rate has not been included within this report.

Overall, the current LWS water rates generate approximately $32 million in rate revenue for the utility. This revenue is used to meet both the operating and capital needs of the system. The City is planning on future rate adjustments in FY 2013/14 and FY 2015/16. This feasibility assessment has taken into consideration those assumed adjustments.

[1] This customer uses at least an average of 1.0 million cubic feet per month, or 7,480,000 gallons per month

3.0 Overview of the Financial Assessment Process

The LWS incurs two types of costs: operating costs and capital costs. Operating costs are related to the operation and maintenance of the system and are typically expensed on an annual basis. In contrast to this, the LWS also incurs capital costs. Capital costs are used to fund capital infrastructure projects. These infrastructure projects may be funded in a variety of ways, but at the most basic level these capital infrastructure projects are typically funded by some combination of rate funding and long-term debt. There are certainly other funding sources (e.g. grants, fees, etc.), but the majority of capital infrastructure projects are funded using rates (cash flow) or the issuance of debt.

In reviewing a utility's rates, a comprehensive rate study typically utilizes three interrelated analyses to address the adequacy and equity of a utility's rates.[2] These three analyses are a revenue requirement analysis, a cost of service analysis, and a rate design analysis. Shown below in Figure 3-1 is an overview of each of these analyses.

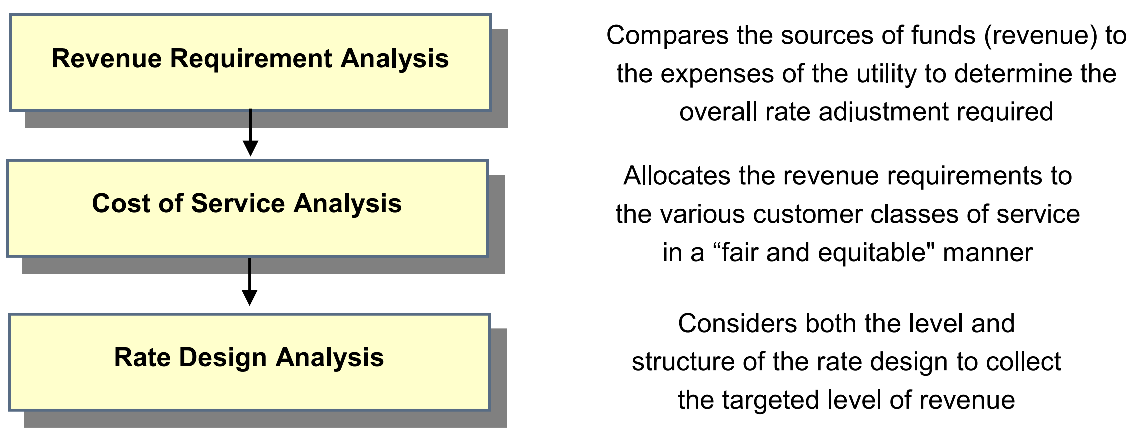


Figure 3-1 Overview of the Comprehensive Rate Study Analysis

The framework shown above contains a revenue requirement analysis, which is a form of a financial needs analysis or a financial assessment. It is the basic analytical framework of a revenue requirement analysis which was used to develop the financial assessment for this study. Analytically, the framework used to develop the revenue requirement analysis is the “cash basis” methodology.

[2] The comprehensive rate study conducted for LWS in 2011 used this analytical framework

The "cash basis" methodology sums O&M expenses, transfer payments, debt service and capital improvement projects funded from rates to equal the total revenue requirement. Figure 3-2 provides a graphical overview of this methodology.

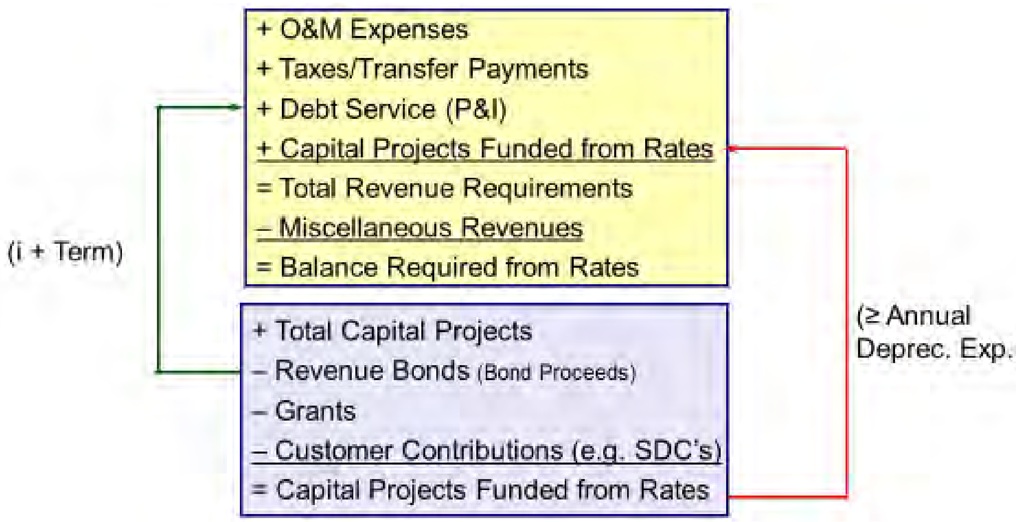


Figure 3-2 Overview of the "Cash Basis" Revenue Requirement Methodology

As can be seen in Figure 3-2, the top (yellow) box is the development of the revenue requirement analysis, while the bottom (blue) box is the how capital projects may be funded and their interrelationship to the revenue requirements. If debt is used to fund a capital project, it must be repaid within the revenue requirement and the amount of the debt payment is a function of both the interest rate on the debt, but also the term or length of repayment on the debt. Ultimately, some portion of a utility's capital projects must be funded from current rates. A simple financial rule that may be used to judge the adequacy of funding capital projects from rates is the funding of an amount that is at least equal to or greater than annual deprecation expense. Depreciation expense is related to the existing infrastructure in place. While it reflects the current infrastructure in place, depreciation expense does not reflect the replacement cost of an existing facility. Hence, the need for funding which may be greater than the annual depreciation expense of the utility.

This basic framework was used in the development of this financial assessment. While the actual analysis is more detailed, the graphical representation shown in Figure 3-2 provides the needed understanding of the basic approach used.

While the basic framework was applied to this study, the financial assessment developed herein also incorporated other key financial planning issues. These included the following:

- DSC [bond covenants]
- Meet a minimum requirement ≥ 1.30

- Capital Projects Funded From Rates
- Used for renewal and replacement projects; ≥ annual depreciation expense
- Maintenance of Adequate Reserves
- Operating Reserve (Working Capital) [90 days of O&M expenses ≅ $4.5 million]
- Construction (Capital Replacement) Reserve [1 year of average replacement costs ≅ $8 million]
- Impact Fee Reserve [no minimum balance]
- Bond Reserves [specific to bond issue and bond covenants – typically equal to one year of debt service payments (average or maximum); often funded in the debt issuance]

Given this basic understanding of the analytical framework used to develop the financial assessment, the focus can shift to the capital projects contained in the Master Plan and the funding tools available to the LWS to fund these projects.

4.0 Capital Projects and Funding Tools

The importance of capital project funding can not be understated. At the present time, the costs within the water utility industry are currently being driven by the cost of infrastructure. In particular, the funding of renewal and replacement related capital projects is a financial challenge for many utilities. The past deferral of these renewal and replacement projects to help minimize rates has created a huge backlog of projects for many utilities. The funding of this backlog of projects, while at the same time funding other growth-related and regulatory-related projects, has created the need for rate adjustments which often exceed inflationary levels.

A key objective in capital planning and creating a financial assessment is to develop a plan which provides adequate funding to construct the needed projects, while at the same time, using funding tools, sources and financing techniques which can help to minimize costs and rates through time. In general, some combination of funding from rates and long-term debt is used to meet that key objective.

In capital planning, it is important to note that there are different types of capital infrastructure projects. These types of capital projects may be categorized as follows:

- Renewal and Replacement Related Projects
- Expansion (Growth) Related Projects
- Regulatory/Reliability Related Projects

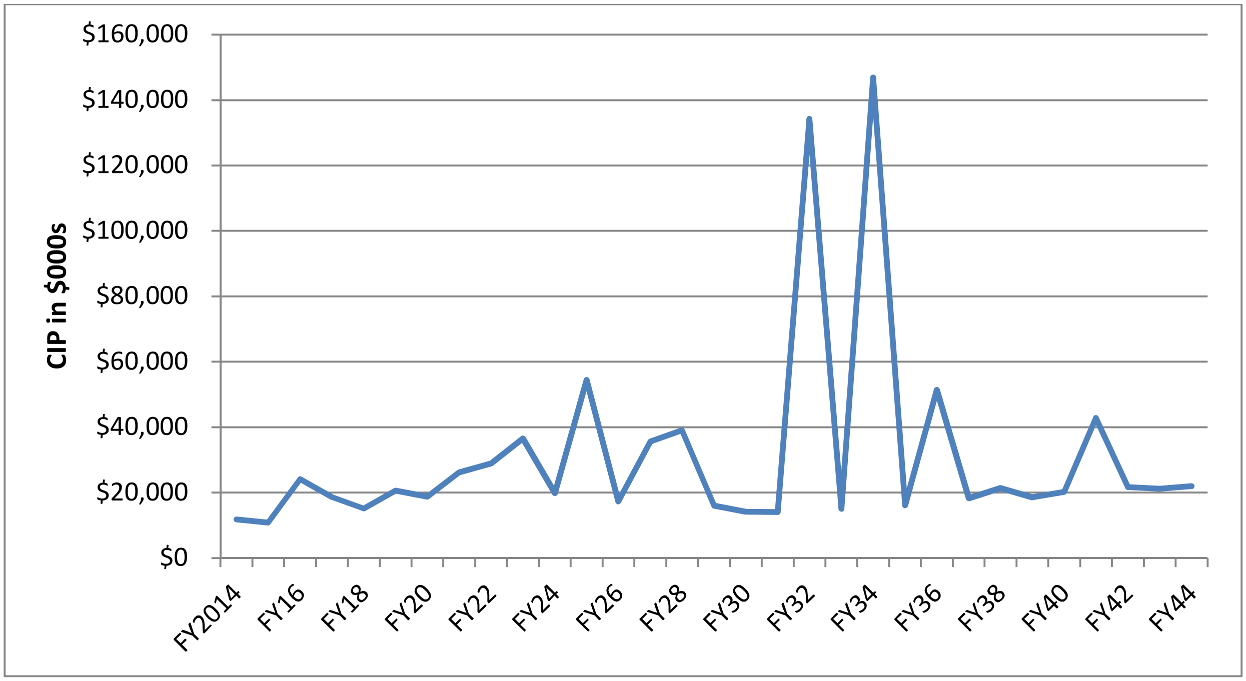
The reason for segregating capital projects into these three basic categories is that each type of capital project may be funded in a slightly different manner. Renewal and replacement projects are related to the replacement of existing infrastructure and ideally should be funded from existing rates.

In contrast to this, an expansion or growth-related project is related to the development of expanded capacity to serve new customers or existing customers with expanded needs. Ideally, these types of projects are funded through growth-related fees. A growth-related project may be funded using long-term debt, but the annual debt service payments may be made from impact fees or impact fee reserves to better match the timing of growth on the system to the annual debt service payments of the expansion project.

Finally, certain projects may be related to meeting a regulatory requirement or creating greater system reliability. These projects are often funded via long-term debt, depending upon the magnitude or size of these projects (e.g. new treatment process at water treatment plant).

While the above discussion has “neatly” segregated the various types of capital projects, the reality is much more complex. For example, an 8” distribution main may be replaced with a 12” main. This would be a combination of a replacement and expansion-related capital infrastructure project. For purposes of this study, the financial assessment has attempted to annually fund a prudent amount from rates for the renewal and replacement projects. The balance of the projects in any year are being funded from other funding sources, which is primarily long-term debt and construction (capital) reserves.

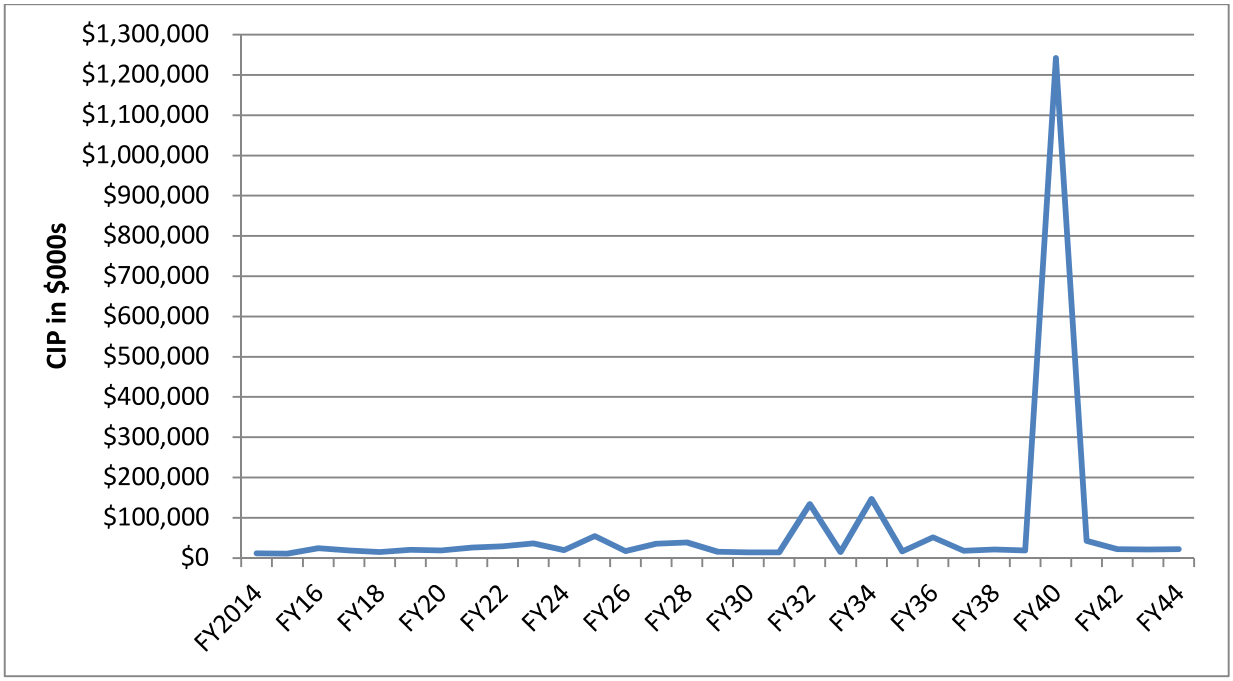
One of the major challenges in developing a capital funding plan and the resulting financial assessment is the size and timing of the capital projects. In any particular year, the capital plan may have a minimal amount of projects, or conversely, a major capital project which greatly exceeds the annual funding for capital projects. The development of capital funding plan and resulting financial assessment must be developed in such a way that it “levels out” the annual funding and minimizes the needed annual rate adjustments. Shown below in Figure 4-1 is the Master Plan annual capital projects.



Note: Excludes the Missouri River Project ≅$500 million (2013$)

Figure 4-1 Master Plan Annual Capital Improvements ($000)

As can be seen in Figure 4-1, the level of annual expenditures ranges from roughly $10 million per year to almost $150 million. Obviously, there are some relatively major capital projects in the later years of the plan. While this graphical summary of the capital projects appears challenging from a funding perspective, when the Missouri River Project is added in, it becomes even more financially challenging. Figure 4-2 provides a graph of the same capital projects as Figure 4-1, but includes the Missouri River Project.



Note: Includes the Missouri River Project ≅$500 million (2013$)

Figure 4-2 Master Plan Annual Capital Projects Including the Missouri River Project ($000)

Figure 4-2 clearly illustrates the impact of the Missouri River Project when placed in the context of the other capital projects within the Master Plan.

As noted previously, this financial assessment has attempted to review the capital projects from three different time perspectives. These are as follows:

- 10-Year Perspective (FY 2014 – FY 2023)
- 30-Year Perspective (FY 2014 – FY 2044) excluding the Missouri River Project
- 30-Year Perspective (FY 2014 – FY 2044) including the Missouri River Project

The 10-year perspective is the most important and critical at this time. The longer-term 30-year perspective provides an important look going forward, but projecting costs 30 years into the future is less predictable and reliable.

5.0 Looking Ahead 10-Years; FY 2014 – FY 2023

The first perspective or time period reviewed was the first 10 years or FY 2014 – FY 2023. To review this time period and provide a financial assessment, HDR Engineering, Inc. (HDR) developed a financial planning model specifically for this task. The model is a simplified version of a revenue requirement model. Provided below is a discussion of the general approach used to develop the financial assessment and the key assumptions, along with a summary of the findings and conclusions.

5.1 Overview of the General Approach

As noted previously, the financial assessment utilized a "cash basis"[3] methodology for the general analytical framework. Next, the financial assessment used LWS's current funding levels as a starting point. The 10-year assessment, as included within this subsection, has excluded the Missouri River Project from funding consideration.

5.2 LWS's Current Funding Level

As noted above, the financial assessment started with LWS's current funding levels. LWS's budget for 2014 was used as this starting point. LWS currently has approximately $32.8 million in revenues. In comparison to this, the utility has approximately $18 million in operation and maintenance (O&M) expenses. This leaves approximately $14.8 million for capital expenditure needs. Of this amount, it is assumed that approximately $10 million is available for rate funding of capital projects. Next, LWS has approximately $5.4 million in current debt service obligations. This is off-set, in part, by impact (growth-related) fees. This off-set has been minimized to avoid over-reliance upon these growth-related fees. Finally, the model has added a component for change in working capital. This component may be used to build up reserves or be drawn down to off-set needed rate adjustments. In this case, reserves were adjusted by $398,000 to balance to LWS's existing revenue sources.

From this framework and starting point for revenues and expenses, projections were made for the following nine years to provide a financial assessment for FY 2014 – FY 2023. A more detailed discussion of this process and key assumptions are discussed below.

5.3 Projection of Revenue and Expenses

The projection of revenues and expenses from the 2014 starting point is the next step of the financial assessment. The key assumptions used to project these revenues and expenses are discussed in more detail below.

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[3] The use of the term "cash basis" should not be confused with accounting terminology. In this particular case, "cash basis" or the "cash needs" methodology is a generally accepted rate setting term.

5.3.1 Projection of Revenues

Rate revenues were calculated based on the 2013 LWS statement of revenues; it was then multiplied by 3.75 percent to reflect the rate increase in November 2013 and used as the target for FY 2014 revenues. After the initial year, the revenues are the product of the revenue of the previous year and an assumed annual customer/rate revenue growth factor of 0.5 percent/year for the first ten years and 0.8 percent/year thereafter. While customer growth is expected to be slightly greater than this assumed level, in recent years the water utility industry has experienced fairly significant declining per capita consumption. There are likely a number of various factors leading to this result, but most importantly, this trend of declining per capita consumption has negatively impacted revenue levels. Thus, customer growth and resulting revenue growth do not exactly correlate. The revenue growth factors used within this financial assessment are assumed to be more conservative (somewhat less) than the anticipated overall customer growth. Projected rate revenues (assuming no rate adjustments) are a function of customer growth and per capita consumption. Rate revenues were projected to be approximately $32.2 million in 2014 and by FY 2023 (assuming no rate adjustments) the rate revenue is projected to be $33.7 million based on these customer growth assumptions.

Other miscellaneous revenues are included as a revenue source, but in this case are relatively minor. For FY 2014, the total revenue available to offset the operating, capital, and debt requirements for the LWS totals $32.8 million and is projected to increase to $34.3 million at the end of the ten year period.

5.3.2 Projection of Expenses

Given the projection of revenues the focus shifts to the projection of expenses. Four main cost components were reviewed in developing the financial assessment:

- Operations and Maintenance (O&M) Expenses
- Annual Debt Service Payments (P + I) [Existing and Future Obligations]
- Capital Improvement Projects Funded From Rates
- Change in Working Capital (Reserves)

The key assumptions associated with each of these cost components are discussed in more detail below.

Operation and Maintenance Expenses – LWS's FY 2014 budget was used as the starting point for the projection of O&M expenses. To simplify the analysis, the O&M expenses were projected using escalation factors to reflect the assumed inflationary change over this time period. The assumed escalation factors included the following:

- Labor – escalated 3% per year

- Medical – escalated 6 percent per year
- Benefits – escalated 2 percent per year
- Materials & Supplies, Equipment, and Miscellaneous – escalated 3 percent per year
- Power – escalated 3.5 percent per year

O&M expenses in FY 2014 were budgeted at $18.1 million. Using the assumed escalation factors, the O&M expenses were projected to increase to $23.6 million by FY 2023. It should be noted that in projecting O&M expenses, no extraordinary O&M expenses were assumed, or changes in staffing levels.

Debt Service (Existing and Future) – The City currently has outstanding debt issues and these debt obligations (annual debt service payments) are included within their FY 2014 budget. There are currently four outstanding loan and bonds for the City: three revenue bonds (2009, 2012, and 2013 issues), and a Nebraska Department of Environmental Quality (NDEQ) loan issued in 2013. The combined annual debt service on the existing (outstanding) debt is approximately $5.4 million.

To fund the future capital projects identified in this plan, this financial assessment assumes that approximately 30 percent of the total capital projects within this 10-year time period will be funded using long-term debt. All new debt is assumed to be a 30-year issue at 5.0 percent interest. During this period, LWS may have access to low-interest loans to fund certain capital projects; however, access to this funding source is not assured and may be of limited amounts. Therefore, assuming a debt service cost which is comparable to the cost of a current revenue bond was viewed as a more conservative approach from a financial planning perspective. At the same time, while the interest cost of debt is currently assumed to be 5.0 percent, this cost can certainly change over time and potentially increase the overall borrowing costs of the LWS. Ultimately, that change in debt costs may impact the financial feasibility of the Master Plan capital projects. However, a simple sensitivity analysis of the assumed interest cost and repayment period for long-term debt was conducted which showed that it certainly had a financial/rate impact, but not to the point of becoming financially “unfeasible”.

Capital Improvements Funded from Rates – As was noted in the general discussion of the funding of capital projects, a simple financial guideline for funding of renewal and replacement capital projects is the need to fund an amount at least equal to or greater than annual depreciation expense. While a recommended minimum level of rate funding for capital projects (renewal and replacement) would be annual depreciation expense, the reality is that an amount equal to the LWS’s annual depreciation expense does not fully fund replacement capital projects. Actual replacement cost is typically at least 1.5 to 2.0 times the system’s depreciation expense. Depreciation expense may reflect the cost of an item placed in service, on average, approximately 15 years ago, assuming a 30-year useful life.

LWS's current depreciation expense is about $7.5 million. Given that, the LWS is funding above the assumed "minimum" levels at this time. For purposes of projecting this replacement funding in the future, the level of CIP funding from rates was gradually increased over time at a rate of 3.5 percent per year. This approach is greatly simplified for purposes of this assessment analysis, but it does continue to recognize the need for this important funding component and continues to enhance the level of funding over time. Funding in FY 2014 is $10 million, gradually increasing to approximately $14.5 million by FY 2023.

This increased funding allows for more projects to be funded on a "pay-as-you-go" basis (i.e. cash funded), while at the same time enhancing the LWS's debt service coverage ratio. This aspect of the analysis and the financial assessment is discussed in more detail below.

Change in Working Capital – The change in working capital component is primarily used to maintain adequate operating reserves and fund construction (capital) reserves. As the O&M expenses increase, the working capital reserves must be maintained at the desired minimum level. In addition, given the magnitude of the capital plan, pre-funding construction (capital) reserves helps minimize long-term borrowing. By pre-funding construction reserves, the funds can be utilized at those points in time when large or significant projects may occur.

Debt Service Coverage (DSC) – When revenue bonds are issued, they often contain certain legal requirements or rate covenants which the utility must meet. A debt service coverage ratio is typically a rate covenant of a revenue bond and it requires the utility to maintain their rates at a sufficient level to assure repayment of the bond (debt). The bondholders judge or test the utility's ability to repay the bond via a debt service coverage ratio test. Simply stated, a debt service coverage ratio is a comparison of net income before debt service payments compared to the total debt service on the revenue bond, or more likely, on all of the utility's outstanding debt service. At a minimum, the utility should maintain a debt service coverage ratio of 1.25 or 1.30 for revenue bonds. However, a DSC ratio of 1.50 is considered a stronger financial target. For purposes of this financial assessment, it was assumed that the LWS must meet or exceed a DSC of 1.30. This means that the utility, after meeting its O&M expenses, has 30 percent more net revenue available to pay debt, than the actual debt service payment (e.g. $1.3 million available to pay a debt payment of $1.0 million).[4]

5.4 Review of the Capital Projects – FY 2014 – FY 2023

The Master Plan provides the capital projects for the 10-year period. Capital improvement projects during the first ten years, from FY 2014 through FY 2023, total approximately $235

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[4] This is a very simplified statement and example. LWS's actual bond covenants should be reviewed for the specific calculation and requirements of the debt service coverage ratio test.

million. Figure 5-1 provides an overview of the size and timing of the capital plan during this 10-year period. As can be seen in Figure 5-1, the capital projects in any particular year range from approximately $10 million to slightly over $40 million. Given that LWS is currently funding approximately $10 million per year for capital projects, it is clear that the funding of the larger projects in the Master Plan will likely need to be debt funded.

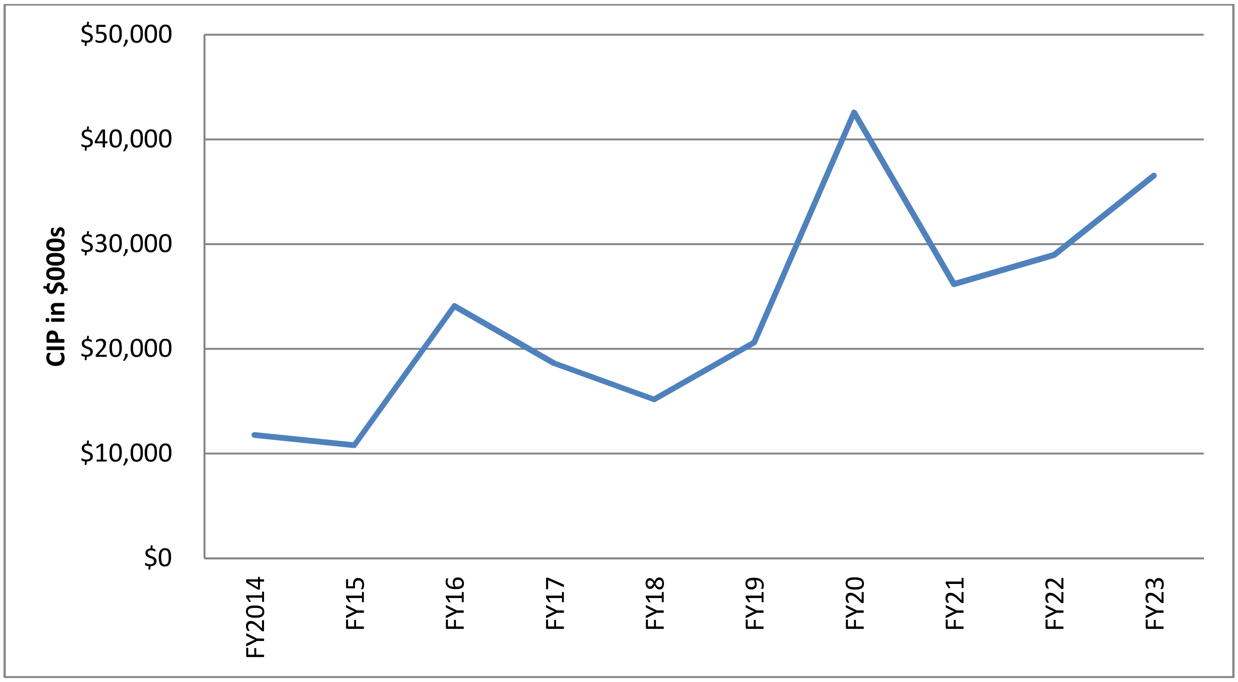

Figure 5-1. Summary of the 10-Year Capital Improvements ($000)

Figure 5-2 provides an overview of the capital plan funding sources for the FY 2014 – FY 2023 time period.

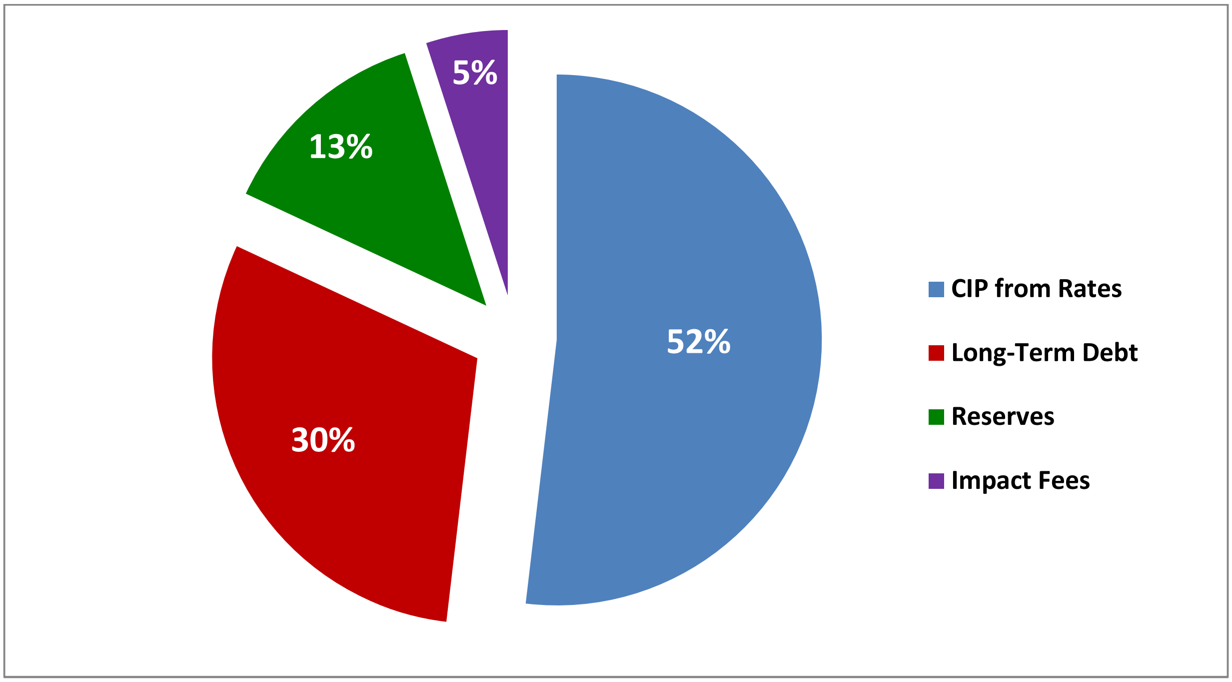

Figure 5-2 Summary of CIP Funding Sources FY 2014 – FY 2023

This funding plan assumes that approximately 52 percent of the projects will be funded on a “pay-as-you-go” basis (i.e. rates/reserves). The funding plan also assumes the need to issue approximately $70 million in new debt.

It should be noted that the City had already planned on future debt issues during this 10-year period and those planned issues are a part of the projected $70 million in new debt issues.

This is a relatively conservative funding plan in that roughly 70 percent of the funding is from LWS funding sources and only 30 percent is from outside funding sources (i.e. long-term debt). It is important to note that if the size or timing of the capital plan changes, the funding plan should be reviewed and adjusted accordingly.

5.5 Annual Debt Service

The assumed issuance of additional debt service has an impact upon the annual debt service payments. In developing this financial assessment, there was no attempt to optimize the timing or size of debt issues. Rather, it is simply presumed that debt is issued in the needed year and the associated debt service payment occurs in the following year. Figure 5-3 presents a summary of the existing and projected future debt service payments for the period of FY 2014 – FY 2023.

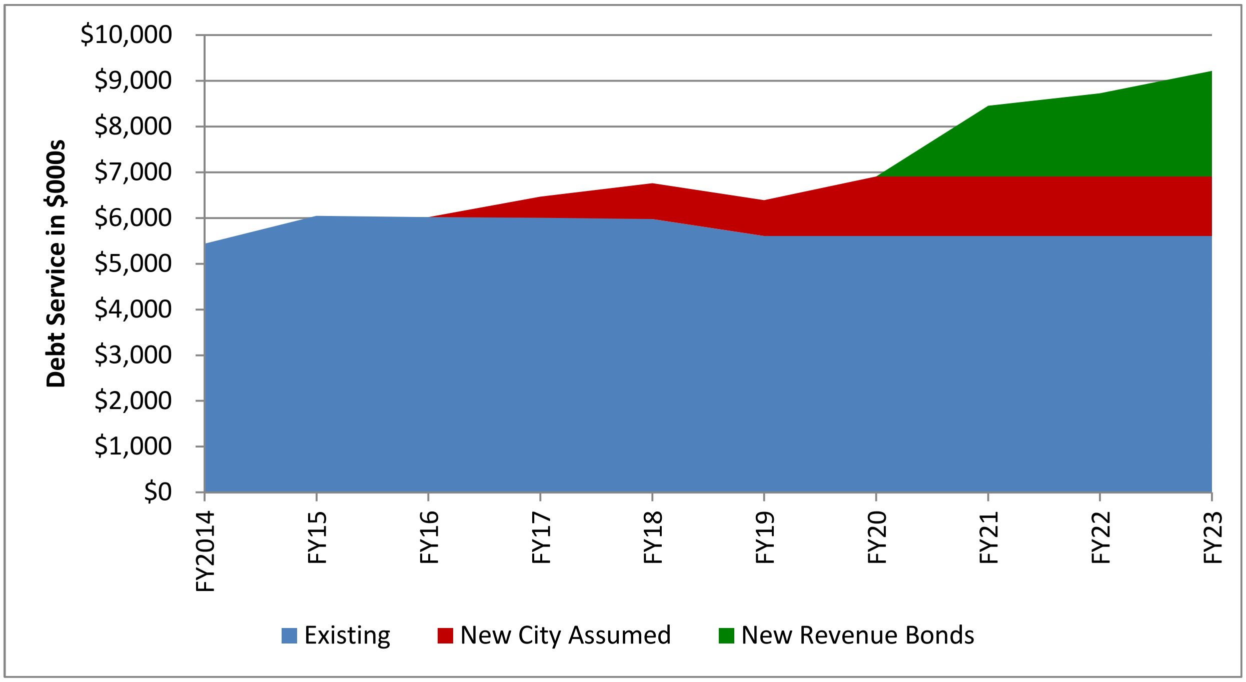

Figure 5-3 Summary of the Annual Debt Service Payments ($000)

Figure 5-3 indicates that the overall annual debt service payments will increase during this 10-year period from approximately $5.0 million per year to slightly over $9.0 million. As noted previously, the future debt issues are assumed to be 30-year issues at an interest rate of 5.0 percent. If LWS issues debt for a shorter term (e.g. 20 years) or at a higher interest rate, the financial impacts will adjust accordingly and be higher than currently shown.

5.6 Summary Financial Assessment – FY 2014 – FY 2023

The financial assessment indicates that the total revenue requirement in FY 2014 is approximately $32.8 million. Using the assumptions discussed above the total revenue requirement increases to approximately $52.8 million by FY 2023 with the issuance of approximately $70 million in new debt.

While the overall increase in the revenue requirements appears substantial, the needed rate adjustments to support the financial assessment average approximately 5 percent per year over the 10-year period. While this level of adjustment is above the assumed inflationary levels, it is not excessively above the assumed general inflation levels of roughly 3.0 percent. As noted previously, this financial assessment is not a formal rate study and not intended to establish or set rates, but rather judge financial feasibility and potential affordability.

Figure 5-4 presents a graphical summary of the financial assessment and the various cost components. Figure 5-4 illustrates that while O&M is certainly increasing over this projected 10-year period, the key drivers are the capital plan and the needed increase in rate funding of capital projects and the increase in debt service from the issuance of additional long-term debt

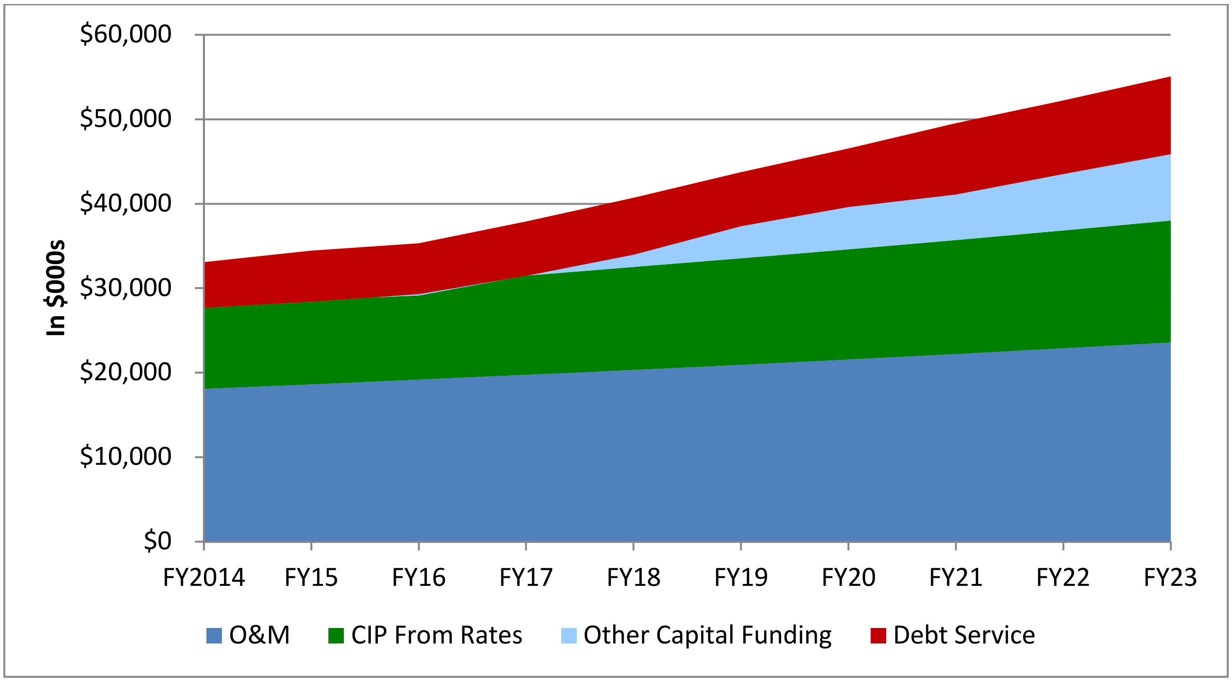

Figure 5-4 Summary of the 10-Year Financial Assessment Components ($000)

5.7 Estimated Residential Bill Impacts from the Financial Assessment

While the anticipated level of the rate adjustments appears to be reasonable, another important perspective is the potential customer bill impacts to a typical residential customer. Figure 5-5 provides a graphical summary of the potential average residential bill impacts. This is shown in both nominal (inflated) and real or net present value (NPV) dollars.

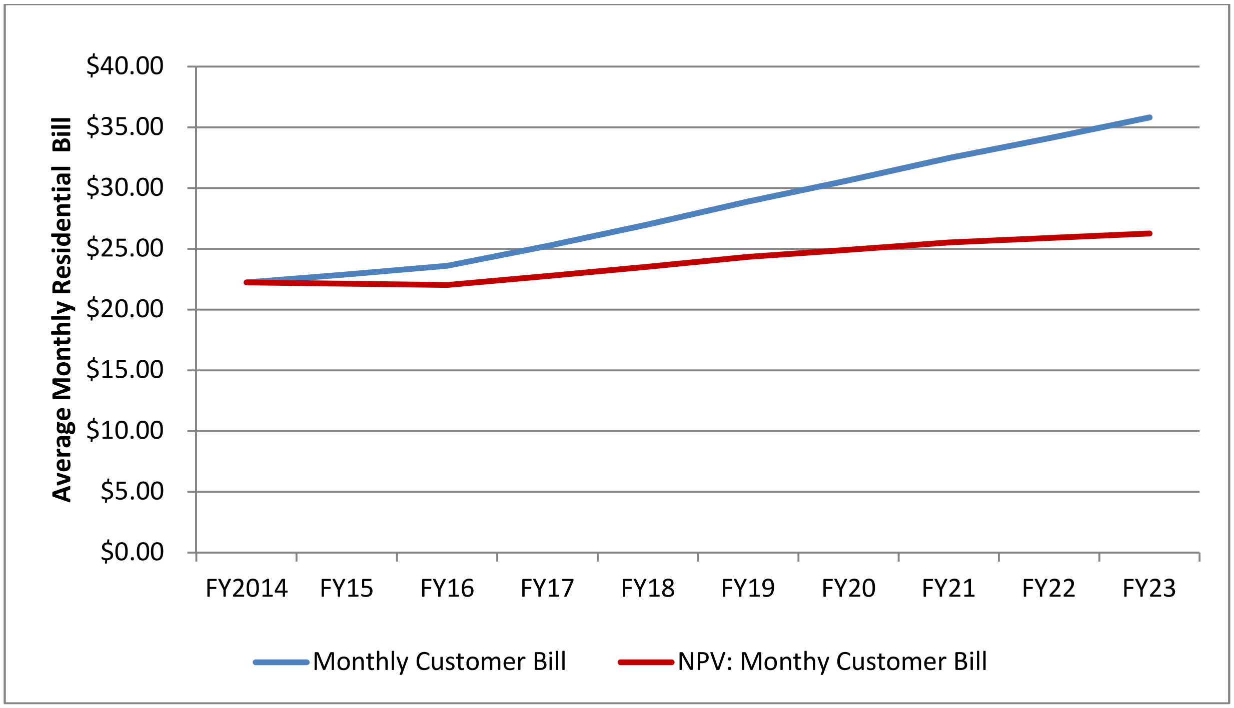

Figure 5-5 Projected Average Residential Monthly Bill – FY 2014 – FY 2023

At the present time, the average monthly residential bill is approximately $22.24. Based on the revenue needs for the 10-year period, the average monthly residential bill could increase to approximately $35.80/month. If this value is adjusted (deflated) for an assumed time value of money, in present day dollars, the impact is $26.27/month in FY 2023. The assumed discount rate used for the present value analysis was 3.5 percent.

5.8 Conclusions for the Financial Assessment for the 10-Year Period of FY 2014 – FY 2023

This time period includes some large and fairly significant capital projects. The financial assessment, as developed herein and using the assumptions discussed within this subsection, has demonstrated a number of items in which certain conclusions can be reached. These include the following:

- The LWS will need to issue additional long-term debt to support the Master Plan capital projects shown for FY 2014 – FY 2023. The financial assessment has indicated that approximately 30 percent of the capital projects, or approximately $70 million of the total capital projects, may need to be funded from new (additional) long-term debt.
- The majority of the funding for the Master Plan capital projects will be funded on a "pay-as-you-go" basis using rate revenues. This will require the LWS to continually increase

the level of funding of the CIP from rates component over this 10-year period from the current level of approximately $10 million per year to about $15 million per year in FY 2023.

- The needed rate adjustments to support the Master Plan averages approximately 5.4 percent per year over this time period. The LWS and the City Council would likely need to adjust rates on an annual basis. However, rate adjustments slightly exceed inflationary levels appear to be affordable when placed in the context of $/month impacts to an average residential customer.
- To support this Master Plan, it has been assumed that the issuance of additional debt will fund a portion of capital projects. If the City issues additional long-term debt, LWS's rates will need to be adjusted to meet debt service coverage requirements.

In summary, the financial assessment developed for FY 2014 – FY 2023 appears to be financially viable and affordable. However, as costs change, along with the cost of borrowing, LWS should review this financial assessment to confirm that it remains feasible.

6.0 Looking Ahead 30-Years; FY 2014 – FY 2044

The second time period reviewed within the financial assessment was a 30-year view or FY 2014 – FY 2044. To review this longer time period, the same analytical framework as the 10-year financial assessment discussed above was used and simply extended the analysis out to FY 2044. Provided below is a discussion of the general approach and key assumptions for this 30-year analysis and assessment, which does not include the Missouri River Project.

6.1 Overview of the General Approach

The general approach used for the 30-year financial assessment was identical to that used for the 10-year assessment. This 30-year financial assessment has excluded the Missouri River Project from funding consideration.

6.2 Projection of Revenue and Expenses

Similar to the 10-year model, the 30-year model extended the 10-year model out an additional 20 years. The general approach and assumptions for the initial 10-year period remained essentially unchanged, but there were some minor changes to the assumptions beyond the first 10-years.

6.2.1 Projection of Revenues

Rate revenues were projected at a growth/escalation factor of 0.5 percent/year for the first ten years. Beyond the first 10 years, rate revenues were escalated at 0.8 percent/year, a slight increase over the first 10 years. Other miscellaneous revenues were escalated and projected at 0.5 percent/year for the first six years and 1.0 percent/year for the remaining years.

6.2.2 Projection of Expenses

The projection of O&M expenses, debt service and capital improvements funded from rates utilized the same assumptions as the 10-year assessment and extended these assumptions for the post FY 2023 time period.

The change in working capital component was again used to smooth out the funding of capital infrastructure. At all times, the 30-year financial assessment met the minimum reserve and debt service coverage requirements.

6.3 Review of the Capital Projects – FY 2014 – FY 2044

The Master Plan provides the capital projects for the 30-year period review. Capital improvement projects during this time period, excluding the Missouri River Project, total approximately $1.0 billion. This represents the inflated (escalated) cost of the projects and is not stated in current (present value) dollars. Figure 6-1 provides an overview of the size and timing of the capital plan.

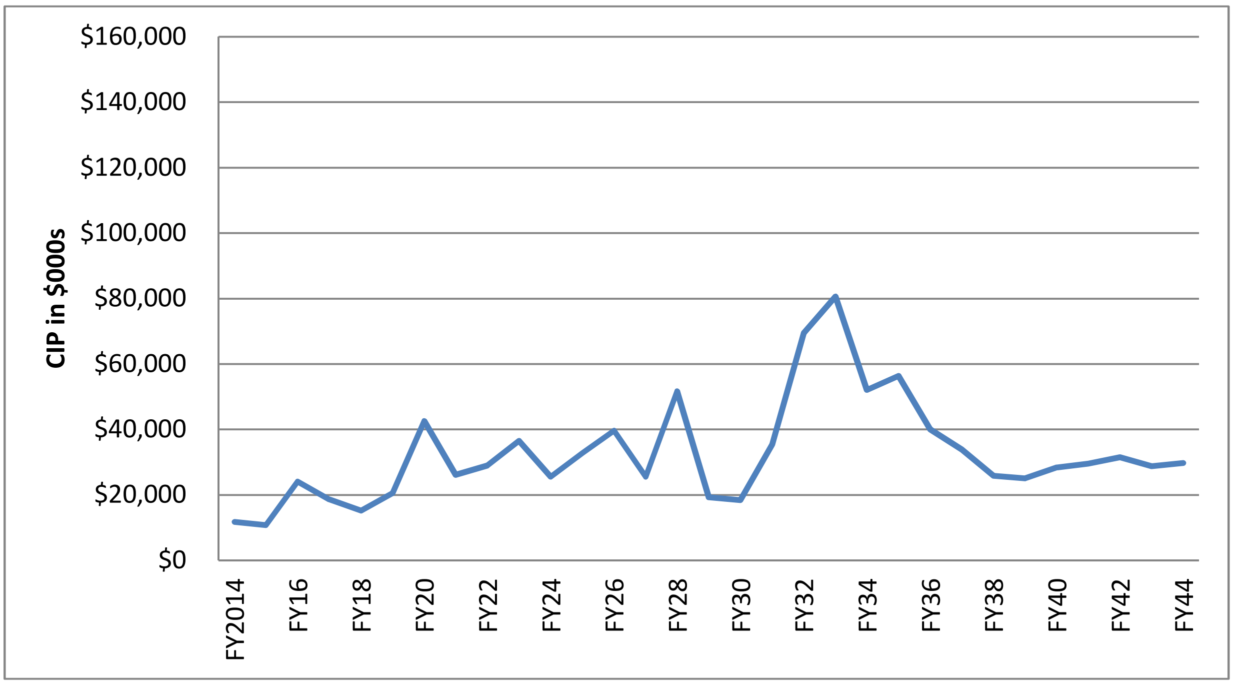

Figure 6-1 Summary of the 30-Year Capital Improvements Excluding the Missouri River Project ($000)

The capital projects, in any particular year, range from approximately $10 million to slightly over $80 million. Figure 6-1 uses a capital plan that has been "smoothed" to level out some of the larger projects. These larger projects were funded over a multi-year period of two to three years to help minimize funding impacts. Figure 6-2 provides an overview of the capital plan funding sources for the FY 2014 – FY 2044 time period.

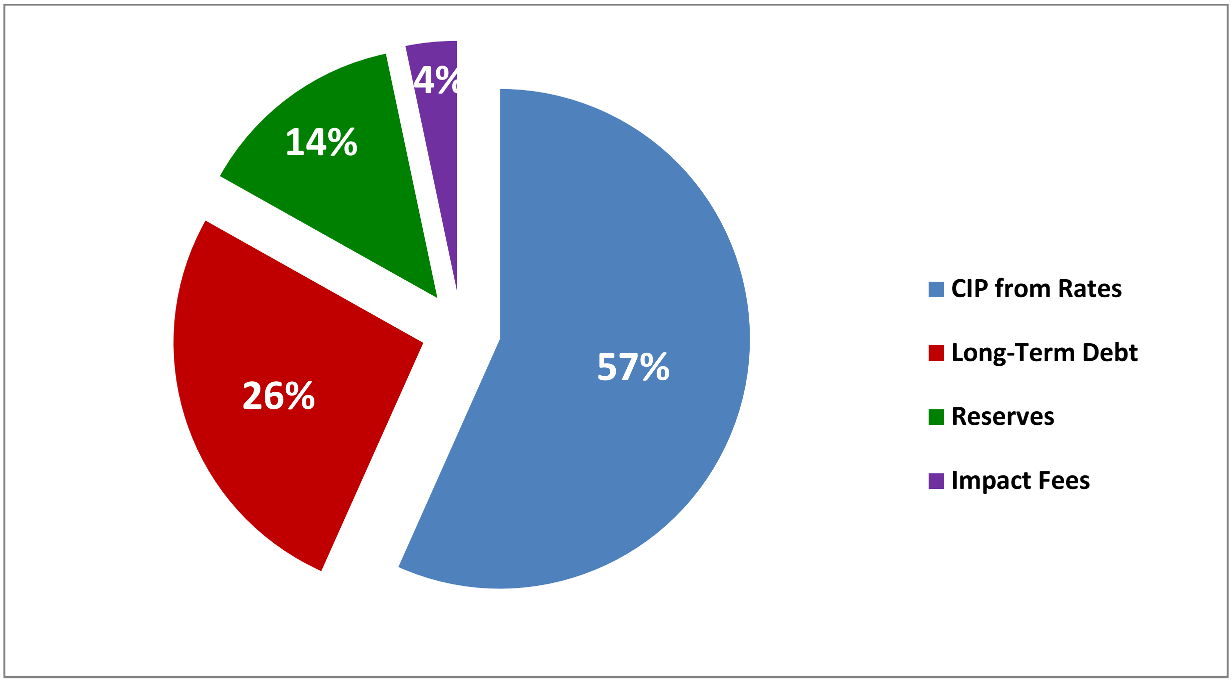

Figure 6-2 Summary of CIP Funding Sources FY 2014 – FY 2044 Excluding the Missouri River Project

The 30-year funding plan assumes that approximately 57 percent of the projects in the Master Plan will be funded on a "pay-as-you-go" basis. The plan also assumes the need to issue approximately $268 million in new debt over this 30-year period. In appearance, this funding plan for the 30-year period uses a mix of funding which is very similar to the 10-year assessment. Again, this funding plan may be viewed as relatively conservative in that it has not been overly reliant upon the use of long-term debt to fund the capital plan.

6.4 Annual Debt Service

The assumed issuance of additional debt service has an impact upon the annual debt service payments. Similar to the 10-year assessment, in the development of this financial assessment, no attempt was made to "optimize" the timing or size of any needed debt issues. Rather, the financial assessment assumes that debt is issued in the needed year of construction and the associated debt service payment begins in the following year. As was noted previously, the assumed interest rate for the long-term debt was 5.0 percent with a 30-year payback period. As these assumptions change over time, the associated debt service payments will change accordingly. Provided below in Figure 6-3 is a summary of the existing and projected future debt service payments for the period of FY 2014 – FY 2044.

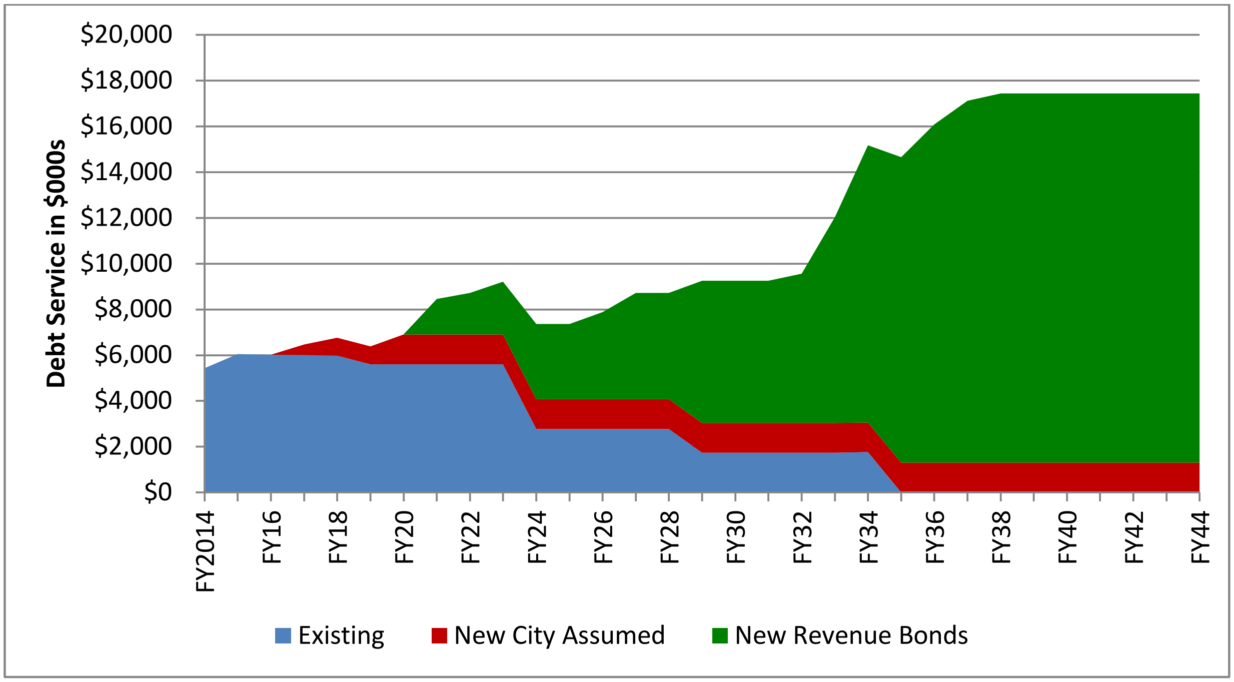

Figure 6-3 Summary of the Annual Debt Service Payments ($000) Excluding the Missouri River Project

Some interesting information can be obtained from Figure 6-3. First, the LWS's existing debt issues and associated debt service payments (blue area) will gradually be reduced and eventually eliminated over this 30-year time period. However, payments on new debt issues will not only replace the older debt service payments, but the expected future payments will exceed the current funding levels.[5] Figure 6-3 indicates that the annual debt service will increase overall during this 30-year period from approximately $5.0 million per year to slightly less than $18.0 million.

As noted previously, the future debt issues are assumed to be 30-year issues at 5.0 percent interest. If LWS issues debt for a shorter term (e.g. 20 years) or at a higher interest rate, the financial impacts will adjust accordingly and be higher than currently shown.

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[5] It is often presumed that when a debt issue is fully paid (retired), the portion of rates previously dedicated to the debt payment can be used to increase pay-as-you-go funding. In this case, future (new) long-term debt issues will exceed the current funding level for debt service and require additional funding for debt service payments.

6.5 Summary Financial Assessment – FY 2014 – FY 2044

From the projection of revenues and expenses, along with a funding plan for the capital projects, a summary of the revenue requirements or financial assessment was developed. Provided below in Figure 6-4 is a summary of the financial assessment developed for the LWS for FY 2014 – FY 2044.

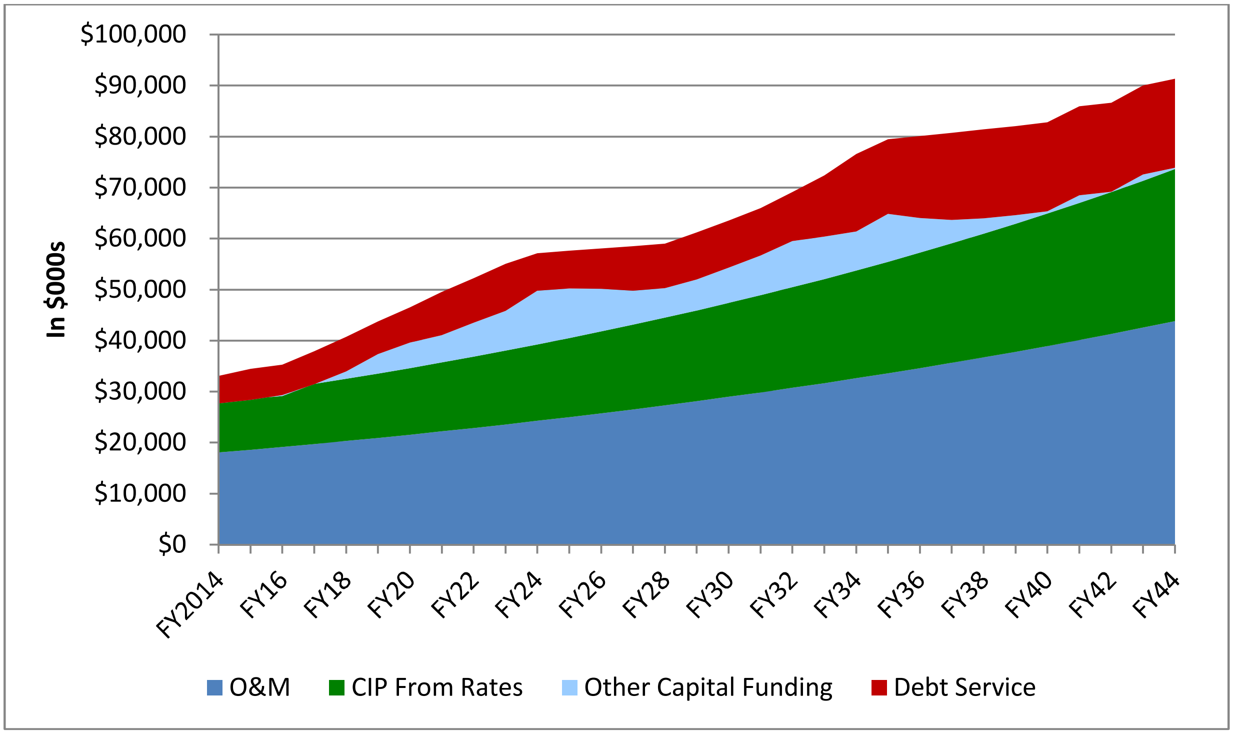

Figure 6-4 Summary of the 30-Year Financial Assessment ($000) Excluding the Missouri River Project

Figure 6-4 has graphically summarized the financial assessment for the 30-year period of FY 2014 – FY 2044. It indicates that the total revenue requirement in FY 2014 is approximately $32.8 million and using the assumptions discussed above, increases to approximately $90.3 million by FY 2044. The need for this level of increase in the revenue requirements is due in part to assumed inflation, but more importantly to the needed funding for capital improvements, along with the increased debt service from the issuance of approximately $268 million in new debt.

While the magnitude of the dollars presented in Figure 6-4 seem exceptionally large in relation to today's costs, the financial assessment is stated in inflated dollars and reflects costs 30 years into the future. The financial assessment assumes needed rate adjustments averaging 3 percent per year or less in the latter years.

6.6 Estimated Residential Bill Impacts from the Financial Assessment

While the level of the rate adjustments discussed above appears to be reasonable, another important perspective is the potential customer bill impacts to a typical residential customer. Figure 6-5 provides a graphical summary of the potential average residential bill impacts under this 30-year financial assessment. The average residential bills are shown in both nominal and real dollars.

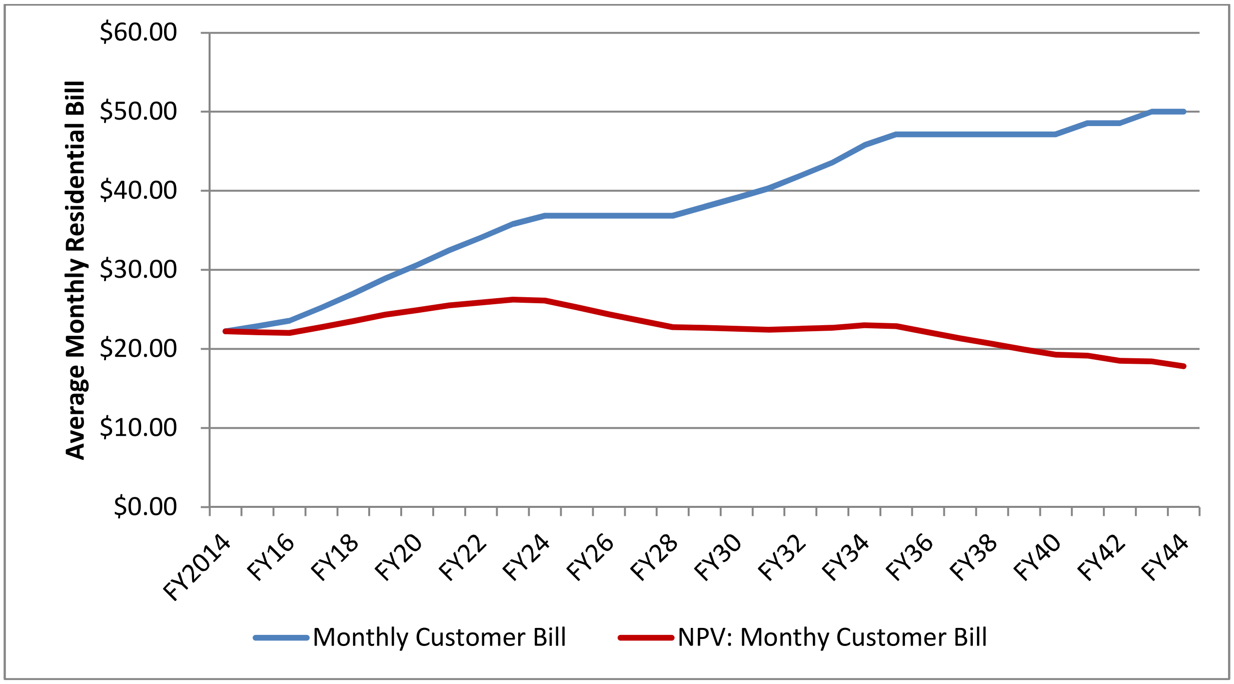

Figure 6-5 Projected Average Residential Monthly Bill – FY 2014 – FY 2044 Excluding the Missouri River Project

At the present time, the average monthly residential bill is approximately $22.24. Assuming the annual rate adjustments shown in Figure 6-4, the average monthly residential bill could increase to approximately $50.01/month by FY 2044. If this value is adjusted (deflated) for the assumed time value of money, then, in present day dollars (NPV) the impact is approximately $17.82/month. The assumed discount rate used for the present value analysis was 3.5 percent.

6.7 Conclusions for the Financial Assessment for the 30 Year Period of FY 2014 – FY 2044

The 30-year time period includes a number of large and fairly significant capital projects. The financial assessment, as developed herein and using the assumptions discussed within this subsection, has demonstrated a number of items in which certain conclusions can be reached. These include the following:

- The City will need to issue additional long-term debt to support the Master Plan capital projects shown for FY 2014 – FY 2044. The financial assessment has indicated that approximately 26 percent of the capital projects or approximately $268 million of the total capital projects over this time period may be funded from new (additional) long-term debt.
- The majority of the funding for the Master Plan capital projects will be funded on a "pay-as-you-go" basis using rate revenues. This will require the LWS to continually increase the level of funding of the CIP from rates component over this 30-year period from the current level of approximately $10 million per year to almost $30 million per year.
- Change in working capital funding will be used to additionally supplement the CIP from rates funding. While the level of funding for construction reserves varies from year-to-year, this may require funding of up to $9.0 million in certain years to fund major capital projects. These major capital projects occur in the mid-2030's.
- The needed rate adjustments to support the Master Plan average approximately 3 percent which is approximately what is required for inflation. LWS and the City Council would likely need to adjust rates on an annual basis.
- To support this Master Plan, it has assumed that the issuance of additional debt will fund a portion of capital projects. If the City desires to issue additional debt, LWS's rates will need to be adjusted to meet debt service coverage requirements.

In summary, the financial assessment developed for FY 2014 – FY 2044, which excludes the Missouri River Project, appears to be financially viable and affordable. However, as costs change, along with the cost of borrowing, the LWS should review this financial assessment to confirm that the assessment remains feasible.

7.0 30-Year Projection with the Missouri River Project

The prior subsection indicated that the Master Plan capital projects for the 30-year period of FY 2014 – FY 2044 appeared to be financially feasible. During this time period, it was assumed that the LWS system would have approximately $1.0 billion in capital projects. The Missouri River Project has a current (estimated) cost of approximately $500 million and when escalated to the end of the 30-year period, it approaches $1.2 billion. When placed in this context, the Missouri River Project essentially doubles LWS's capital plans over this 30-year time period.

A project of this magnitude raises a number of serious financial questions. Most importantly is whether this project is "affordable" and if so, is there a financial strategy that LWS should consider for this particular project.

7.1 Establishing a Missouri River Project Financial Strategy

There are no simple strategies to fund a project of this magnitude. However, there are several strategies to be considered.

As soon as possible, begin to set aside funds in a “dedicated” Missouri River Project reserve. The intent of this reserve is to begin to pre-fund the project such that it does not require 100 percent debt financing in 2040 or when built. A significant amount of funds will need to be collected annually and set aside into this dedicated reserve. Even with this set aside, LWS may be able to fund only 10 percent to 15 percent of the total expected project costs from this reserve.

The intent of funding this reserve is two-fold. First, it sets aside funds for the project, but more importantly, it begins to ramp LWS’s rates up to a point to be able to support the eventual debt service payments associated with the project. Once the project is built, the financial strategy is that LWS will have gradually built into the rates, over the last 30 years, an amount that will pay a substantial portion of the annual debt service payment going forward. The key to this strategy is it should minimize the need for a major rate adjustment (e.g., a doubling of rates in a single year) at the time the debt is issued.

When the project is being built, LWS should deplete the dedicated Missouri River Project reserve and apply those reserves against the project. The balance of any needed funds to construct the project will be obtained from the issuance of long-term debt.

While this strategy appears to be relatively sound on the surface, it will likely be more complicated in reality. In particular, asking today’s customers to fund a project that is potentially 30 years into the future and may or may not be built creates a certain set of political challenges on its own. Though not impossible, it may be difficult to start the reserve in the near future and instead, LWS may need to wait until there is greater certainty around the project. However, that strategy has its own pitfalls in that the amount of funds collected in the dedicated project reserve may be minimal due to the shortened amount of time available to accumulate funds. Alternatively, the size of the rate adjustments needed over the shorter time period to ramp up to the anticipated level of debt service may be too daunting.

This financial assessment developed herein for the Missouri River Project is only intended to answer the basic question of whether it is potentially feasible to be funded. To review this issue, a 30-year analysis was utilized that included the Missouri River Project within the capital plan. Provided below is a discussion of the general approach and key assumptions.

7.2 Overview of the General Approach

The general approach used to review the Missouri River Project used the 30-year financial assessment previously discussed above. This 30-year financial assessment has included the

Missouri River Project for funding consideration. In doing so, the model was adjusted to include the annual funding for the dedicated Missouri River Project reserve. These funds were set aside and eventually used at the assumed time of construction to off-set the total costs of the project.

7.3 Projection of Revenue and Expenses

The financial assessment which includes the Missouri River Project used the same assumptions as the 30-year financial assessment to project the revenues and expenses.

7.4 Projection of the Capital Plan Including the Missouri River Project

The capital plan for this financial assessment included all the same projects as the 30-year assessment, plus the Missouri River Project. Similar to all other projects, the Missouri River Project was escalated to reflect the time period when it is anticipated to be built. In this case, it is assumed to be constructed at the end of the 30-year period. Figure 7-1 provides an overview of the size and timing of the capital plan when the Missouri River Project is included.

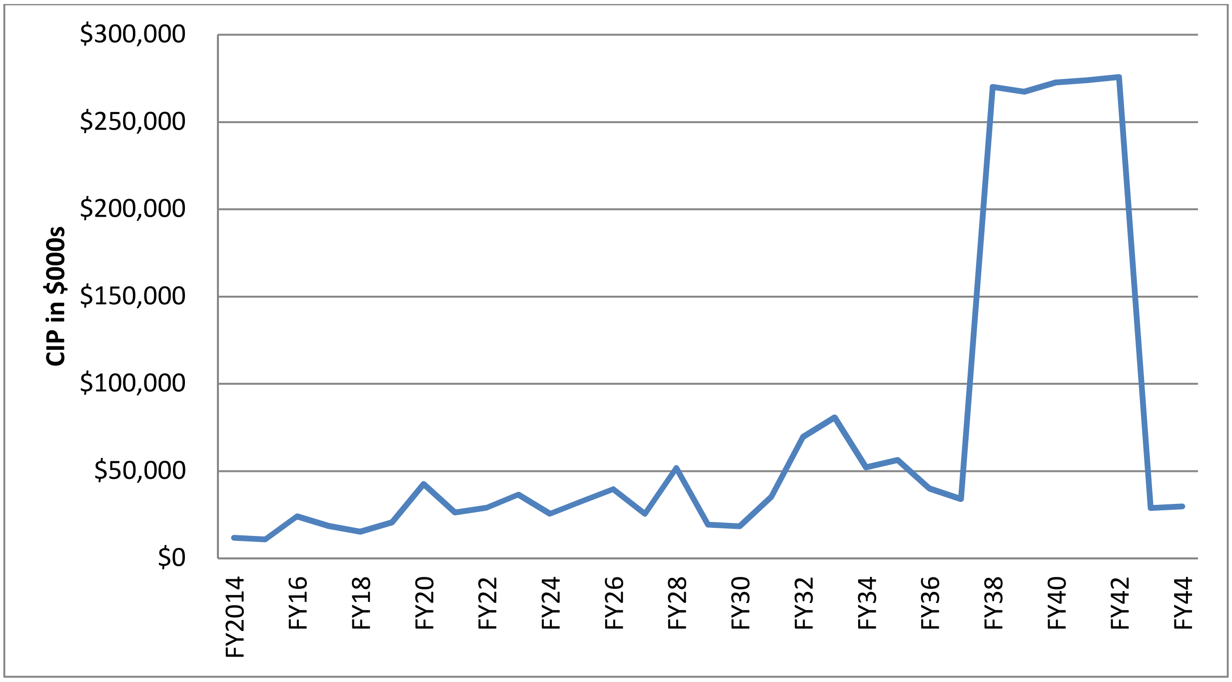

Figure 7-1 Summary of the 30-Year Capital Improvements Including the Missouri River Project ($000)

The Missouri River Project has a significant impact upon the capital plan and all other projects seem to pale in comparison. The Missouri River Project was spread over a 4-year period which translates to an expenditure of approximately $250 million per year.

As noted in the financial strategy section, it was assumed that the utility would begin to fund the Missouri River Project reserve in FY 2018 and begin by funding $1 million per year. Eventually, and over gradual time, the annual contribution is increased to $32 million by FY 2040. At that point, the Missouri River Project reserve will fund approximately $298 million of the project costs. Additional funds will be collected during the construction period from rates and an additional $25 million is assumed to also be available in construction reserves. When taken together, this is approximately 28 percent of the anticipated project cost, meaning that the balance or approximately $882 million (72 percent) will need to be funded from long-term debt.

7.5 Annual Debt Service

Given the magnitude of the Missouri River Project, the impact to annual debt service payments is significant. Figure 7-2 presents a summary of the existing and projected future debt service payments for the period of FY 2014 – FY 2044.

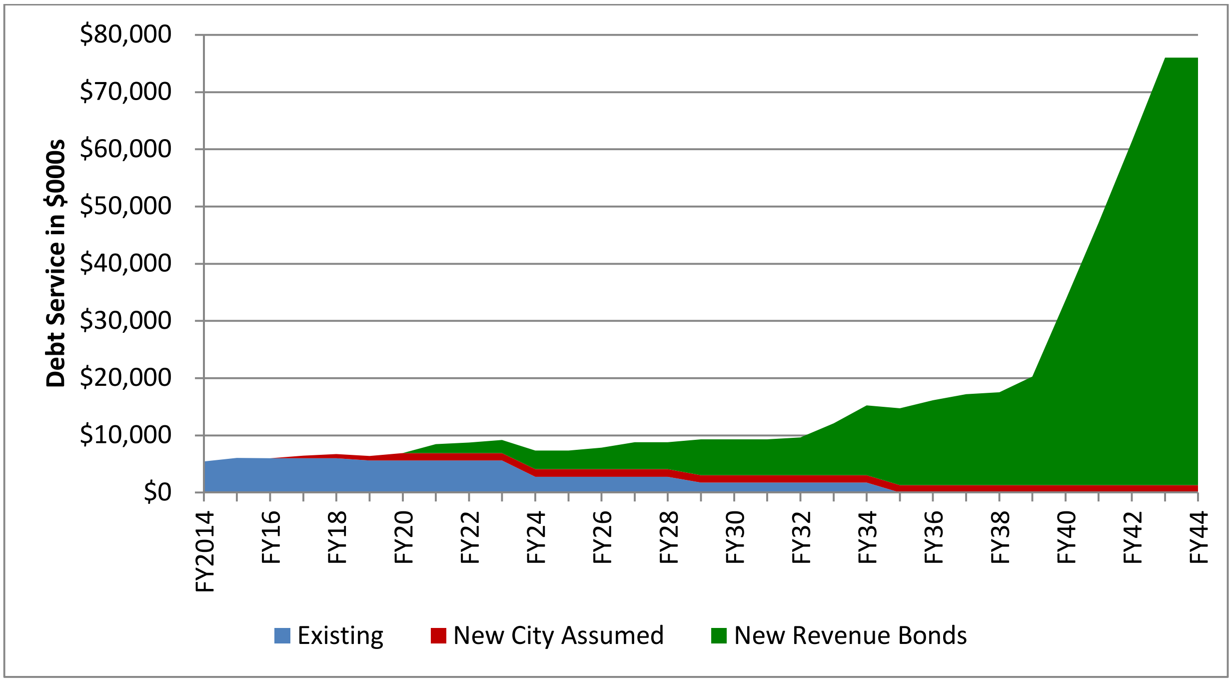

Figure 7-2 Summary of the Annual Debt Service Payments ($000) Including the Missouri River Project

Based on Figure 7-2, the financial impact upon debt payments related to the Missouri River Project is significant. The prior 30-year assessment had projected annual debt service payments in FY 2044 of approximately $18.0 million. In this case, the annual debt service is nearly $75 million per year.

As noted previously, the future debt issues are assumed to be 30-year issues at 5.0 percent interest. If the LWS issues debt for a shorter term (e.g. 20 years) or at a higher interest rate, the financial impacts will adjust accordingly and be higher than currently shown. In this case, the Missouri River Project is particularly sensitive to the debt assumptions given the size and magnitude of the needed borrowing.

7.6 Summary Financial Assessment – FY 2014 – FY 2044

From the projection of revenues and expenses, along with a funding plan for the capital projects, a summary of the revenue requirements or financial assessment was developed. Figure 7-3 presents a summary of the financial assessment developed for the LWS for FY 2014 – FY 2044.

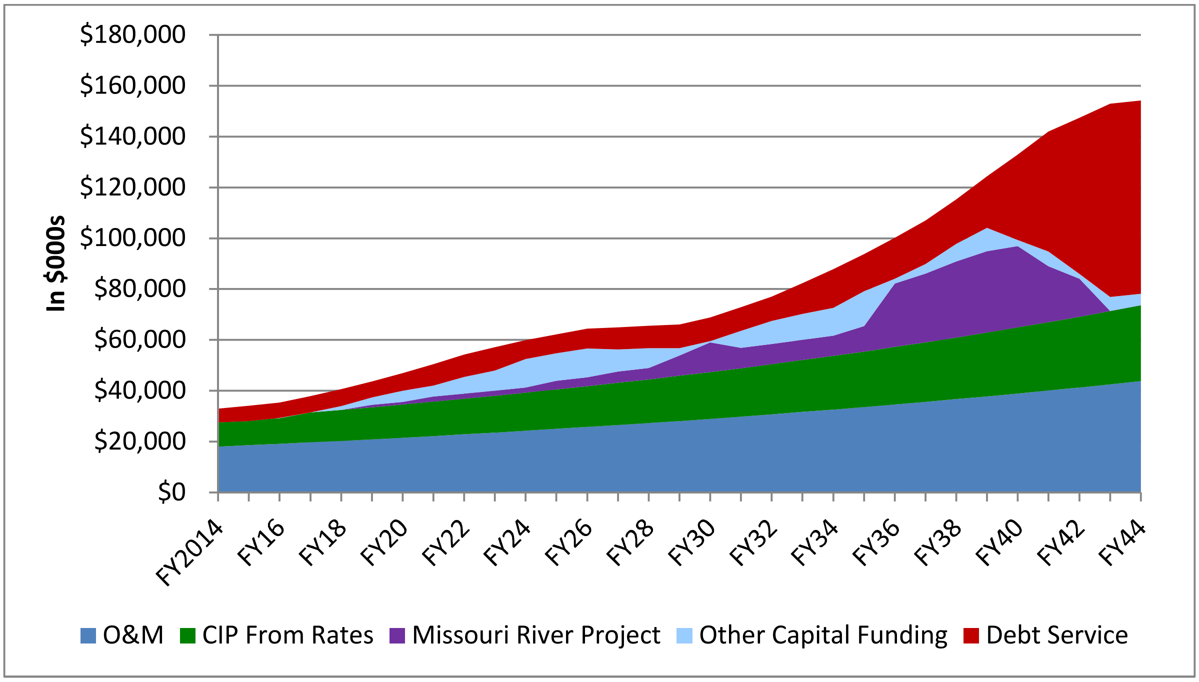

Figure 7-3 Summary of the 30-Year Financial Assessment ($000) Including the Missouri River Project

Figure 7-3 has graphically summarized the financial assessment for the 30-year period of FY 2014 – FY 2044 and includes the Missouri River Project. When one closely examines the components of the revenue requirement, the purple area is the funding of the Missouri River Project reserve. In addition, the light blue funding of the “other capital funding” helps to position the utility for the eventual debt service which occurs (dark red). By FY 2044, and using the assumptions discussed above, the total revenue requirement is approximately $160 million. This is not a doubling of the revenue requirement from the 30-year financial assessment which excluded the Missouri River Project, but it does come close to that.

The financial assessment assumes needed rate adjustments averaging 4.5 percent per year over this time period. The latter years in the analysis are significantly greater than the same time period under the 30-year assessment.

7.7 Estimated Residential Bill Impacts from the Financial Assessment

While the level of the rate adjustments shown in Figure 7-3 appears to be reasonable, as with the other financial assessments, a review of the potential customer bill impacts to a typical residential customer was undertaken. Figure 7-4 provides a graphical summary of the potential average residential bill impacts. This is shown in both nominal and real dollars.

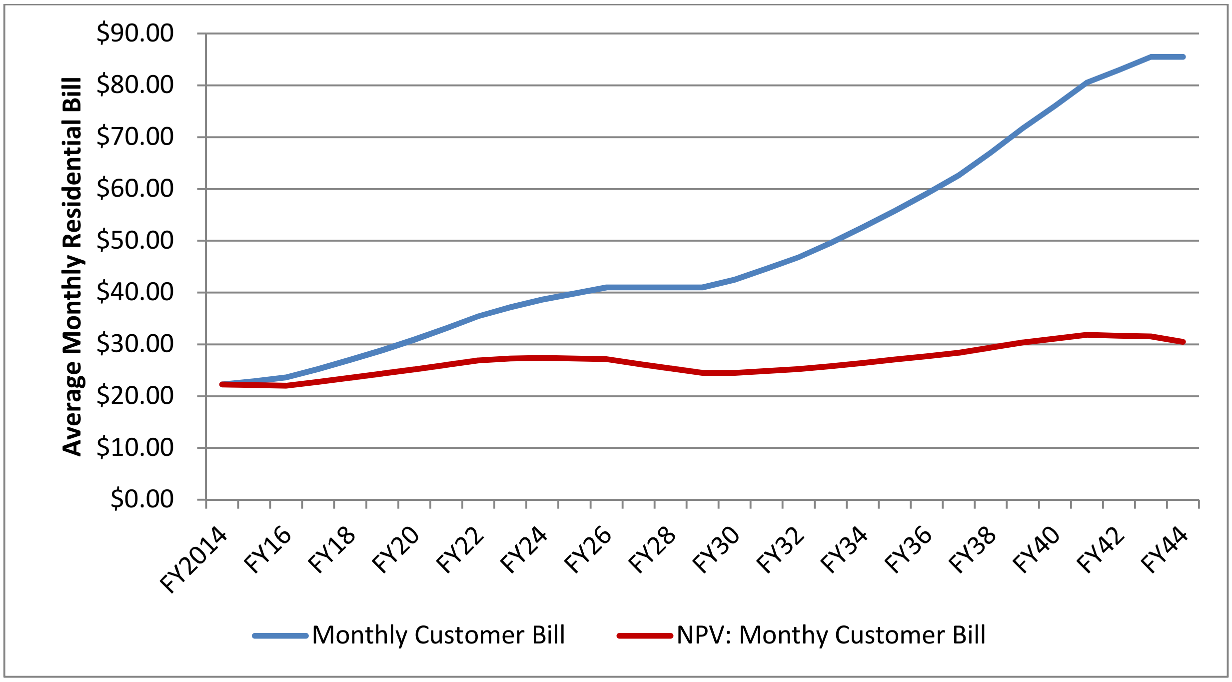

Figure 7-4 Projected Average Residential Monthly Bill – FY 2014 – FY 2044 Including the Missouri River Project

At the present time, the average monthly residential bill is approximately $22.24. Assuming the annual rate adjustments shown in Figure 7-3, the average monthly residential bill could increase to approximately $85.49/month. If this value is adjusted (deflated) for the assumed time value of money, then, in present day dollars the impact is approximately $30.46/month. The assumed discount rate used for the present value analysis was 3.5 percent.

As with the other 30-year analysis, this result is subject to the variability of the assumptions used over the 30-year period, and the result may vary significantly under different assumptions.

7.8 Conclusions for the Financial Assessment for the 30 Year Period of FY 2014 – FY 2044 Which Includes the Missouri River Project

The Missouri River Project is of such a magnitude that it has a significant financial impact upon the utility as a single stand-alone project. The financial assessment, as developed herein and using the assumptions discussed within this subsection, has demonstrated a number of items in which certain conclusions can be reached. These include the following:

- The City will need to develop a specific funding and financing strategy for the Missouri River Project should it become a viable project.
- The project is potentially 30 years out into the future. Given that, it is difficult to project costs that far into the future with any degree of confidence. There are too many variables, known and unknown at this time, which would allow HDR to confidently state at this time that the Missouri River Project is financially feasible. However, under the assumptions used herein, it would potentially appear to be financially feasible.
- The funding/financing strategy used herein may not be politically feasible or palatable. The funding strategy developed herein requires significant pre-funding of the project; well before the project comes on line and provides service. There may be significant pushback from the community regarding this funding strategy.
- Assuming the funding strategy is deemed acceptable and politically palatable it will require significant financial discipline on the part of the LWS and unified long-term support by the City Council to adjust rates on an annual basis, at levels often exceeding general inflation, to support the funding of the dedicated reserve. Given that this reserve funding may need to occur over a 20 to 30 year time period, it may be difficult for the LWS to maintain City Council support given that makeup and viewpoints of Council members will certainly change over time.
- If a dedicated reserve is created for the Missouri River Project, it must be restricted and used only for the Missouri River Project.
- For the Missouri River Project to proceed, LWS will need to financially position themselves to be able to support debt issues of this magnitude.
- Other non-traditional funding sources may need to be considered. For example, a dedicated sales tax may be considered to obtain a portion of the funding. However, this additional sales tax would likely need to be voter approved.
- Ultimately, this project may need additional partners or outside project funding assistance (e.g., grants, low-interest loans, etc.). The impact of this would likely enhance the financial feasibility of the Missouri River Project.

In summary, the financial assessment developed for FY 2014 – FY 2044, which included the Missouri River Project, may be financially viable and affordable. In this financial assessment, the Missouri River Project was potentially made to be viable and affordable through the use of a specific funding strategy and the issuance of significant levels of long-term debt. It is important for the LWS to understand that the preliminary conclusions reached concerning the Missouri River Project hinge upon the assumptions used herein and as those assumptions may change over the course of time, so might the financial viability of this particular project. From this analysis, one should not conclude that that Missouri River Project is financially feasible under all circumstances. Rather, one should conclude that the Missouri River Project is potentially financially feasible, but in order to be feasible, LWS will need to develop a clear funding/financial strategy for the project and have the discipline to properly execute that financial strategy and plan over the long term.

8.0 Review of Affordability Issues

Affordability is a concern of all utilities given the fact that rates and charges for utility services have been increasing at a pace which exceeds the cost of living (CPI). Affordability has now come to the forefront of many discussions, particularly as it relates to major capital infrastructure funding and financing.

When discussing utility rates and customer bills it is not uncommon to consider a customer's *ability* and *willingness* to pay for utility services. Willingness to pay is related to the perceived value of the commodity and is not the focus of the affordability discussion. Rather, ability to pay is focused on whether customers have sufficient income to pay for the service.

Ability to pay and affordability has traditionally been measured around median household income (MHI) levels. Under this approach, affordability for the community is defined as a percentage of the MHI. Average residential bills which exceed this threshold are considered "unaffordable". Typical measures used have ranged from 1.5 percent to 2.5 percent of a community's MHI. A simple example assuming a 2.0 percent threshold and a community with a MHI of $45,000 would have an affordability threshold of $900 per year or $75.00 per month. In other words, if a typical (average) residential bill exceeded $75.00 per month, then the rate for the community would be considered "unaffordable". However, one of the drawbacks to this approach is that MHI is a community-wide measure, and even if a rate is defined as "affordable" for the community that does not necessarily imply that all customers can afford the rate. Income levels will vary within a community and each community will have some segment of their population which may have affordability issues.

There currently is much discussion within the utility industry concerning these types of affordability measures. The Environmental Protection Agency uses a two-phase approach to assess financial capability (affordability). The first phase assesses the impact on the household (similar to the above example), while the second phase examines the debt, socioeconomic and financial conditions of the utility. The results of this two-phase analysis are combined into a Financial Capability Matrix.

The financial capability calculation is fairly detailed and it is not the intent of this Master Plan report to evaluate affordability impacts at that level. However, with regard to the range of values used for the analysis, EPA assesses the impact to communities as follows[6]:

Financial Impact	Residential Indicator (% of MHI)
Low	Less than 1.0% of MHI
Mid-Range	1.0% - 2.0% of MHI
High	Greater than 2.0% MHI

In the case of the City the MHI is approximately $46,600. Using the 2.0 percent measure, this means that the average residential bill would need to be $77.00 per month before the rate would be considered “unaffordable” on a community wide basis. Stated an alternative way, the current average residential bill of $22.24 is approximately 6/10 of 1 percent, which is in the low financial impact range. Given the currently low rates that LWS has, it would seem that nothing within this review of the Master Plan and the development of the Financial Assessments would indicate that the Master Plan is unaffordable on a community-wide basis.

While the above has concluded that on a community-wide basis there does not appear to be affordability issues, at an individual level there may be affordability issues. As LWS’s rates continue to increase over time, the LWS and the City may consider different means or methods to address the needs of these specific customers.

9.0 Summary

This section of the Master Plan has reviewed the issue of financial feasibility of the Master Plan and in particular the capital plans contained within the Master Plan. Financial assessments were developed using generally accepted financial planning and rate setting methodologies. The findings and conclusions from our analyses indicate that the capital plans within the Master Plan are financially feasible into the near future (10-years) and potentially over the longer-term (30-years).

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[6] Source: EPA: Combined Sewer Overflows – Guidance for Financial Capability Assessment and Schedule Development, February 1997.

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Lincoln Water System Facilities Master Plan

Chapter 9 - Summary of Recommendations

May 2014

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Table of Contents

List of Tables

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Abbreviations and Acronyms

CCI	Construction Cost Index
CIP	Capital Improvements Program
City	City of Lincoln
CMMS	Computerized Maintenance Management System
ENR	Engineering News-Record
GIS	Geographic Information Systems
HAA5	Haloacetic Acids
HCW	Horizontal Collector Well
LPlan 2040	Lincoln/Lancaster County 2040 Comprehensive Plan
LWS	Lincoln Water System
Master Plan	2013 Facilities Master Plan
mgd	Million Gallons per Day

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1.0 Introduction

1.1 General

The overreaching principle of the Facilities Master Plan is to develop a plan to resolve current deficiencies, to meet projected demands, improve water quality, and extend the useful life of assets within a financially viable and affordable framework.

The planning period for this effort is from the year 2012 through the year 2060. Three specific planning intervals have been used.

- 2025 (short-term)
- 2040 (mid-term)
- 2060 (long-term)

These planning periods coordinate with the planning periods of the City of Lincoln's (City) Comprehensive Plan, LPlan 2040. The planning periods are described in *Chapter 2* in detail. A series of recommendations have been developed from the analysis completed within each specific area or chapter which address the following:

- Water Supply
- Water Treatment
- Transmission and Distribution System
- Water Main Replacement
- Asset Management
- Financial Planning

Recommendations are summarized to identify immediate, short-term, mid-term and long-term improvement needs of the water system.

- Immediate improvements are those that have been identified as having a higher priority within the short-term planning period as a result of the immediate need or as a result of currently anticipated conditions.
- Short-term improvements will extend service to the limits of the Tier 1- Priority B area and to meet the projected capacity needs of the expanded service area.
- Mid-term improvements will extend service to the limits of the Tier-1 Priority C area and to meet the projected capacity needs of the expanded service area.
- Long-term improvements will meet the projected capacity needs for the Tier II growth area.

1.2 Cost Estimates

In any engineering study that develops a capital improvement program, it is necessary to make estimates of the project costs required to implement the program. To that end, basic cost data must be obtained or developed for each type of construction and system components laid out in sufficient detail to permit determination of approximate project costs.

Inherently, cost estimates vary depending on the phase of the project when they are developed, which determines the level of detail and the expected accuracy of the estimate. The Association for the Advancement of Cost Engineering International (AACE International) Recommended Practices, specifically Document No. 18R-97, outlines typical cost estimate accuracies based on the overall status of the project.

The total project cost necessary to complete a project consists of expenditures for land acquisition, construction costs, all necessary engineering services, contingencies, and such overhead items as legal, administrative and financing services.

The cost of land acquisition is not included in the project cost presented. Although land or easement acquisition is a significantly activity that determines whether a project occurs, the cost is generally a small portion of the overall program cost.

Construction costs cover the material, equipment, labor and services necessary to build the proposed project. Prices used in this study were obtained from a review of previous reports and pertinent sources of construction cost information. Construction costs used in this report are not intended to represent the lowest prices which may be achieved but rather are intended to represent a median of competitive prices submitted by responsible bidders.

Such factors as unexpected construction conditions, the need for unforeseen mechanical and electrical equipment, and variations in final quantities are a few examples of items that can add to planning level estimates of project cost. To cover such contingencies, an allowance of twenty-five percent (25%) of the construction cost has been included.

Engineering services may include preliminary investigations and reports, site and route surveys, geotechnical and foundation explorations, preparation of design drawings and specifications, engineering services during construction, construction observation, construction surveying, sampling and testing, start-up services, and preparation of operation and maintenance manuals. Overhead charges cover such items as legal fees, financing fees, and administrative costs. The costs presented in this report include a fifteen percent (15%) allowance for engineering services, legal, and administrative costs.

The additional detail on the basis of costs and the detailed cost estimates for each project can be found in the individual chapters of the Master Plan.

In considering the estimates presented, it is important to realize that they are reported in year 2013-2014 dollars, and that future changes in the cost of materials, equipment and labor will cause comparable changes in project costs. A good indicator of changes in construction costs is the Engineering News-Record (ENR) Construction Cost Index (CCI), which is computed from prices of construction material and labor, and based on a value of 100 in the year 1913.

Cost data in this report are based on an ENR CCI (20-city average) of 9547, which is the annual average value for year 2014 (through April). Cost data presented in this report can be adjusted to any time in the past or future by factoring it by the ratio of the then-prevailing ENR CCI (20-city average) divided by 9547.

2.0 Water Supply

The City's existing well field consists of 40 vertical wells, 2 horizontal collector wells (HCWs) with a third HCW currently under construction. A caisson for a future fourth HCW is also currently being installed.

An evaluation was conducted to determine the ability of the existing source of supply to meet the projected water capacity needs of the system. The City's well field, in its current configuration, will not be able to meet projected demands through the planning horizon of 2060. Therefore, an evaluation was conducted of raw water supply alternatives that would increase the raw water capacity of the City to meet both the short-term, mid-term and long-term demands.

2.1 Immediate Need

The Master Plan analysis for well performance identifies wells which should be rehabilitated or abandoned and replaced. Following recommended procedures for well maintenance, the following priority order is recommended for well rehabilitation or replacement.

a. Wells that have a sharp downward trend in specific capacity and have a specific capacity below 50 gpm/ft. .

b. Wells that have a sharp downward trend in specific capacity or have a specific capacity below 50 gpm/ft. .

c. Wells that are operating at less than seventy-five percent (75%) of their original specific capacity.

If any well that exhibits a sharp downward trend in specific capacity does not respond to a standard well treatment, it is recommended that a detailed evaluation of that well be performed by a licensed well driller or pump installer. This evaluation should provide a recommendation for a more aggressive well rehabilitation or for replacement of the well.

2.2 Short-Term Water Supply

Expansion of the existing well field through construction of a fourth HCW is the recommended approach to reduce the short-term deficit between the projected water demands and the water supply capacity of the existing infrastructure. As of the development of the Master Plan, the City has authorized the construction of the caisson for the fourth HCW. Transmission piping, well pumps and the well house will be constructed to meet short-term water supply improvement needs by 2018.

2.3 Mid-Term Water Supply

Based upon analysis presented in *Chapter 3*, the construction of a fifth and sixth HCW is recommended. The first of these two wells or fifth HCW has a projected need to be operational by 2025 and would address projected increase in water demands to approximately 2035. The second of these two wells or sixth HCW has a projected need to be operational before 2035.

2.4 Long-Term Water Supply

The long-term supply alternative evaluated in the Master Plan was a Missouri River Project. The Missouri River is operated as a navigable channel, and the streamflow is regulated from upstream reservoirs. A well field constructed in the Missouri River alluvium would be less susceptible to low streamflow during the summer months when demands for water are highest.

For the purposes of the Master Plan, it was assumed that the long-range alternative would supply a maximum of 60 mgd, which is sufficient to meet the water supply needs of the City through 2060 if the mid-term supply alternative is not developed. The implementation of the mid-term supply option would reduce the initial capacity needs for the long-term alternative. However, development of a 60-mgd supply along the Missouri River would provide the City with a diversified source of supply that is more resistant to drought and could provide opportunities to develop this supply option as a regional water supply.

2.5 Summary of Improvements

The recommended improvements and associated costs for water supply for the short, mid and long-terms are summarized in Table 2.1.

Table 2-1 Opinion of Probable Cost – Water Supply Improvements

Year	Description	Current Cost Basis [1]	Future Cost Basis – 3% Inflation [2]	Future Cost Basis – 5% Inflation [3]
Immediate Projects				
2016	Rehab Wells:	$196,000	$241,000	$227,000
2017-2022	Ongoing Rehab/Replacement of Existing Wells	$300,000	$338,000-$391,000	$365,000-$465,000
Short-Term Projects				
2016	Fourth HCW River Crossing/Bank Stabilization	$4,200,000	$4,600,000	$4,900,000
2016	Equip fourth HCW Cassion with Well House, Pumps and Electrical, Roads/Transmission Piping	$6,100,000	$6,700,000	$7,100,000
Mid-Term Projects				
2024	Construct fifth HCW on East Bank (including roads and transmission piping)	$12,600,000	$17,000,000	$22,000,000
2034	Construct sixth HCW on East Bank (including roads and 48" raw water transmission main)	$24,300,000	$45,000,000	$68,000,000
Long-Term Projects				
2016	Collector Well Investigation – 2 Sites for Missouri River Project	$550,000	$601,000	$637,000
2018	Missouri River Project – Well Field Property Purchase	$2,410,000	$2,800,000	$3,100,000
2040	Missouri River Project	$499,500,000	$1,200,000,000	$1,900,000,000

Notes:

1. *Engineering and Contingency estimates are included in each item at a value of Contingency 25% and Engineering 15% of the item cost.*
2. *Inflated to projected year dollars at 3% per year inflation rate through 2017.*
3. *Inflated to projected year dollars at 5% per year inflation rate for years beyond 2017.*

3.0 Water Treatment

Three phases of capacity expansion improvements for water treatment are recommended throughout the planning period based upon the water demand projections developed in the Master Plan.

3.1 Immediate Need

There are no recommended improvements to the Water Treatment System identified as an immediate need for the short-term.

3.2 Short-Term Water Treatment

Planning for design and construction of a 12 mgd capacity expansion of the West Plant is recommended to be undertaken by Year 2022 and for the improvements to be in service by Year 2027 when the existing 120 mgd capacity is projected to be exceeded.

Elements of the recommended improvements include the following:

- Filter pilot testing of alternative dual-media gradations and configurations to determine the final recommended dual-media system, filtration rate, backwashing procedure, control scheme, and any other related improvements.
- Filter dual-media conversion of all West Plant filters.
- SDS TTHM and HAA5 testing to determine what amount of collector well water could be used in the West Plant while safely maintaining compliance with the Stage 2 DBPR.
- New sodium hypochlorite generation system to replace gaseous chlorine storage and feed system.
- New aqueous ammonia system to replace anhydrous ammonia storage and feed system.

3.3 Mid-Term Water Treatment

Planning for design and construction of an initial 30 mgd capacity expansion of the East Plant is recommended to be undertaken by Year 2029 for the improvements to be in service by Year 2034 when the 132 mgd capacity is projected to be exceeded.

Elements of the recommended improvements include the following:

- One additional ozone contact basin
- Four additional filters
- One additional clearwell

- Additional chlorine storage
- Additional ammonia storage
- Additional polymer storage

3.4 Long-Term Treatment Water

Planning for design and construction of a second 30 mgd capacity expansion of the East Plant is recommended to be undertaken by Year 2047 for the improvements to be in service by Year 2052 when the 162 mgd capacity is projected to be exceeded. As discussed in *Chapter 4*, the timing of the Missouri River Project may impact the need for this second East Plant expansion.

Elements of the recommended improvements include the following:

- One additional ozone contact basin
- Four additional filters
- One additional clearwell
- One additional ozone generator

3.5 Summary of Recommended Improvements

The various recommended improvements for capacity expansion are summarized in Table 3-1 as follows:

Table 3-1 Opinion of Probable Cost – Capacity Expansion Improvements

Year[1]	Description	Current Cost Basis[2]	Future Cost Basis - 3% Inflation [3]	Future Cost Basis - 5% Inflation[4]
2027	12 mgd West Plant Expansion[5]	$14,588,000	$22,043,000	$28,827,000
2034	First 30 mgd East Plant Expansion	$25,200,000	$46,872,000	$70,207,000
2052	Second 30 mgd East Plant Expansion[6]	$23,800,000	$75,589,000	$159,579,000
	Total of All Projects	**$63,588,000**	**$144,504,000**	**$258,613,000**

Notes:
1. *The year listed is when the additional capacity needs to be operational*
2. *2013 dollars.*
3. *Inflated to projected year dollars at 3% per year inflation rate.*
4. *Inflated to projected year dollars at 5% per year inflation rate.*
5. *Testing for the West Plant expansion is required in approximately 2022.*
6. *The timing of the Missouri River Project may impact the need of the second East Plant Expansion.*

4.0 Transmission and Distribution Systems

Based on the findings of the steady state hydraulic analyses, the water age analyses, the fire flow analyses, operational and efficiency analyses, and the Year 2060 long-range plan, an improvement program was prepared for each of the planning periods. This capital improvement program (CIP) includes budget costs and is staged and prioritized to identify reinvestment needs and improvements for additional capacity and reliability through Year 2060. Criteria for recommended improvements to address rehabilitation/replacement projects are identified in *Chapter 6* with the exception of main upsizing for fire flow deficiencies and rehabilitation of two transmission mains.

Improvement mains in undeveloped areas are subject to location change to conform to growth patterns and actual development. Factors that may accelerate or delay improvement mains include availability of right-of-way, scheduling of street improvements, and construction of other utilities.

For residential service it is recommended that the City continue its general policy of installing minimum sizes of 16-inch mains on a one-mile grid and 12-inch mains on half-section alignments, adjusted to accommodate local street patterns. Minimum design standards recommended by Lincoln Water System (LWS) are 6-inch for residential, 8-inch for commercial, and 12-inch for industrial areas.

The short-term recommended improvements will provide service to the limits of Tier I- Priority A and B development areas.

The immediate improvements should be viewed as a subset of the short-term improvements. They are recommended to correct existing deficiencies, and provide a partial list of projects that should be included in the next 6-years of the LWS CIP. Some short-term improvements that are not specifically identified as immediate are prioritized based on known or anticipated development.

The immediate improvements are summarized in Table 4-1. The recommended improvements include the following:

- Yankee Hill Road Main Improvements
- West Vine Street Pump Modifications to Remove Eddy Current Coupling.
- Valve Replacement and Automation at 51st Street Reservoirs and Pumping Station.
- Nebraska Innovation Campus Redundant Supply Immediate Improvement.
- Merrill Street Pumping Station Decommissioning/Demolition.
- Northeast Pump Modifications to Remove Eddy Current Coupling.
- South 56th Street Pumping Station Decommissioning.
- Southeast Pumping Station PRV Vault to High Service Level.

- Control Valve or Similar Water Quality Improvement at Pioneers Pump Station.
- Valve Vault Relocation to “A” Street Reservoirs Site.
- 12.0 MGD Firm (18.0 MGD Installed) Permanent Yankee Hill Pumping Station.
- Cheney Pumping Station Decommissioning/Demolition.
- 3.0 MGD Firm (6.0 MGD Installed) Booster Pumping Station at I-80.
- Improvement Mains for Development north of I-80.
- 8.0 MGD Firm (12.0 MGD Installed) Permanent Northwest 12th Street Pumping Station.
- Northwest 12th Street Pumping Station (Fallbrook) Decommissioning/Demolition.
- Replace 10.1 MGD Pump with 20.2 MGD Pump at East Vine Street Pumping Station to Southeast Service Level.
- Pioneers Pumping Station VFD Additions.
- Tank Mixing Study and Improvements.
- Immediate Distribution System Extensions.
- Immediate Pressure Monitoring Stations.
- Immediate Automatic Flushing Hydrants for Chlorine Residual.
- Immediate Fire Flow Improvements.

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Table 4-1 Recommended Immediate Transmission and Distribution Systems Capital Improvements

Year	Description	Type of Improvement	Current Cost Basis[1]	Future Cost Basis (3%)[2]	Future Cost Basis (5%)[3]
Pumping, Storage and Transmission					
2014	West Vine Street Pump Modifications to Remove Eddy Current Coupling	Pumping	$72,000	$75,000	$76,000
2016	Valve Replacement and Automation at 51st Street Reservoirs and PS	Valving	$324,000	$355,000	$376,000
2020	Merrill Street Pumping Station Decommissioning/Demolition	Pumping	$216,000	$266,000	$304,000
2016	Northeast Pump Modifications to Remove Eddy Current Coupling	Pumping	$72,000	$79,000	$84,000
2016	South 56th Street Pumping Station Decommissioning	Pumping	$75,000	$82,000	$87,000
2015	Valve Vault Relocation to "A" Street Reservoirs Site	Valving	$259,000	$275,000	$286,000
2017	12.0 MGD Firm (18.0 MGD Installed) Permanent Yankee Hill Pumping Station	Pumping	$4,313,000	$4,855,000	$5,243,000
2018	Cheney Pumping Station Decommissioning/Demolition	Pumping	$216,000	$251,000	$276,000
2018	3.0 MGD Firm (6.0 MGD Installed) Booster Pumping Station at I-80	Pumping	$971,000	$1,126,000	$1,240,000

Year	Description	Type of Improvement	Current Cost Basis[1]	Future Cost Basis (3%)[2]	Future Cost Basis (5%)[3]
2019	8.0 MGD Firm (12.0 MGD Installed) Permanent NW 12th Street Pumping Station	Pumping	$2,875,000	$3,433,000	$3,853,000
2020	Northwest 12th Street Pumping Station (Fallbrook PS) Decommissioning/Demolition	Pumping	$216,000	$266,000	$304,000
2019	Replace 10.1 MGD Pump with 20.2 MGD Pump at East Vine Street PS to Southeast SL	Pumping	$1,829,000	$2,184,000	$2,452,000
2019	Pioneers Pumping Station VFD Additions	Pumping	$173,000	$207,000	$232,000
2019	Tank Mixing Study and Improvements	Quality	$575,000	$687,000	$771,000
Distribution					
2014	Yankee Hill Road Main Improvements	Distribution	$4,430,000	$4,563,000	$4,652,000
2015	Nebraska Innovation Campus Redundant Supply Immediate Improvement	Distribution	$860,000	$913,000	$949,000
2016	Southeast Pumping Station PRV Vault to High SL	Valving	$144,000	$158,000	$167,000
2016	Control Valve or Similar Water Quality Improvement at Pioneers Pump Station	Quality	$259,000	$284,000	$300,000
2018	Improvement Mains for Development north of I-80	Distribution	$1,946,000	$2,256,000	$2,484,000

Year	Description	Type of Improvement	Current Cost Basis[1]	Future Cost Basis (3%)[2]	Future Cost Basis (5%)[3]
2014-2019	Immediate Fire Flow Improvements	Fire Flow	$2,076,000	$2,342,000	$2,533,000
2014-2019	Immediate Distribution System Extensions	Distribution	$8,537,000	$9,482,000	$10,164,000
2014-2019	Immediate Pressure Monitoring Stations	Monitoring	$138,000	$156,000	$167,000
2014-2019	Immediate Automatic Flushing Hydrants for Chlorine Residual	Quality	$69,000	$79,000	$85,000
-	Total Immediate Projects	-	$30,645,000	$34,374,000	$37,085,000
Average Cost Per Year				**$5,729,000**	**$6,181,000**

Notes:
1. *Engineering and Contingency estimates are included in each item at a value of Contingency 30% and Engineering 20% of the item cost.*
2. *Inflated to projected year dollars at 3% per year inflation rate.*
3. *Inflated to projected year dollars at 5% per year inflation rate.*

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4.1 Short-Term Improvements

The recommended short-term capital improvements are summarized in Table 4-2. The short-term improvements include the following:

- Parallel Transmission Main from Northeast Pumping Station to Vine Street Reservoirs.
- Cheney to Southeast PRV Station for Water Quality.
- Water Main on Northwest 56th Street.
- Belmont to Low PRV Station.
- Northwest Reservoir (2 MG Elevated) and Pipeline for Northwest Service Level.
- Add 20.9 MGD WTP High Service Pump.
- Nebraska Innovation Campus Redundant Supply Short-term Improvement.
- Adams Street Reservoir (5 MG above-grade) and Pipeline for High Service Level.
- Short-term Distribution System Extensions.
- Short-term Pressure Monitoring Stations.
- Short-term Automatic Flushing Hydrants for Chlorine Residual

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Table 4-2 Recommended Short-Term Transmission and Distribution Systems Capital Improvements

Year	Description	Type of Improvement	Current Cost Basis[1]	Future Cost Basis (3%)[2]	Future Cost Basis (5%)[3]
Pumping, Storage and Transmission					
2020-2022	Parallel Transmission Main from Northeast PS to Vine St Reservoirs	Transmission	$24,840,000	$31,477,000	$36,730,000
2022	Northwest Reservoir and Pipeline for Northwest SL (2 MG elevated)	Storage	$6,799,000	$8,872,000	$10,548,000
2023	Add 20.9 MGD WTP High Service Pump	Transmission	$1,503,000	$2,020,000	$2,449,000
2024	Adams Street Reservoir and Pipeline for High SL (5 MG above-grade)	Storage	$13,285,000	$18,390,000	$22,722,000
Distribution					
2020	Cheney to Southeast PRV Station for Water Quality	Quality	$144,000	$239,000	$274,000
2021	Water Main on NW 56th Street	Distribution	$1,246,000	$1,579,000	$1,841,000
2021	Belmont to Low PRV Station	Valving	$144,000	$183,000	$213,000
2023	Nebraska Innovation Campus Redundant Supply Short-term Improvement	Distribution	$1,127,000	$1,515,000	$1,836,000
2020-2025	Short-Term Distribution System Extensions	Distribution	$22.574,000	$29,934,000	$36,011,000
2020-2025	Short-Term Pressure Monitoring Stations	Monitoring	$138,000	$186,000	$223,000
2020-2025	Short-Term Automatic Flushing Hydrants for Chlorine Residual	Quality	$69,000	$95,000	$112,000
-	**Total Short-Term Projects**	-	**$71,869,000**	**$94,490,000**	**$112,968,000**
Average Cost Per Year				**$15,748,000**	**$18,828,000**

Notes:
1. *Engineering and Contingency estimates are included in each item at a value of Contingency 30% and Engineering 20% of the item cost.*
2 *Inflated to projected year dollars at 3% per year inflation rate.*
3. *Inflated to projected year dollars at 5% per year inflation rate.*

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4.2 Mid-Term Improvements

The recommended mid-term capital improvements are summarized in Table 4-3. The mid-term improvements include the following:

- Transfer Pipeline from Vine Street to “A” Street.
- Add 20.2 MGD Pump at East Vine Street Pumping Station to Southeast Service Level.
- Add 5.0 MGD Pump at Pioneers Pumping Station.
- Transmission Main Replacement from Platte River WTP to "A" Street.
- 40.0 MGD Firm (60.0 MGD Installed) Pump Station at Northeast Reservoir and Pumping Station.
- Belmont Connector Main.
- Replace Pump at WTP with 20.9 MGD Pump.
- Add 6.0 MGD Pump in Yankee Hill Pumping Station for 18.0 MGD Total Firm and 24.0 MGD Total Installed.
- Cheney II Reservoir (3 MG elevated) and Pipeline for Cheney Service Level.
- Mid-Term Distribution System Extensions.
- Mid-Term Pressure Monitoring Stations.
- Mid-Term Automatic Flushing Hydrants for Chlorine Residual.

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Table 4-3 Recommended Mid-Term Transmission and Distribution Systems Capital Improvements

Year	Description	Type of Improvement	Current Cost Basis[1]	Future Cost Basis (3%)[2]	Future Cost Basis (5%)[3]
Pumping, Storage and Transmission					
2027	Transfer Pipeline from Vine St to "A" St	Transmission	$16,256,000	$24,589,000	$32,186,000
2028	Add 20.2 MGD Pump at East Vine Street Pumping Station to Southeast SL	Pumping	$1,743,000	$2,716,000	$3,624,000
2029	Add 5.0 MGD Pump at Pioneers Pumping Station	Pumping	$360,000	$578,000	$786,000
2031	Transmission Main Replacement from Platte River WTP to "A" St	Transmission	$64,032,000	$109,011,000	$154,101,000
2033	40.0 MGD Firm (60.0 MGD Installed) Pump Station at Northeast Reservoir and Pumping Station	Transmission	$12,938,000	$23,368,000	$34,329,000
2037	Replace Pump at WTP with 20.9 MGD Pump	Pumping	$1,503,000	$3,056,000	$4,848,000
2039	Add 6 MGD Pump (18.0 MGD Total Firm/24.0 MGD Total Installed) in Yankee Hill Pumping Station	Pumping	$432,000	$932,000	$1,537,000
2040	Cheney II Reservoir and Pipeline for Cheney SL (3 MG above-grade)	Storage	$8,777,000	$19,497,000	$32,769,000
Distribution					
2035	Belmont Connector Main	Distribution	$3,696,000	$7,082,000	$10,812,000
2033	Mid-Term Distribution System Extensions	Distribution	$27,667,000	$49,970,000	$73,409,000
2033	Mid-Term Pressure Monitoring Stations	Monitoring	$138,000	$250,000	$367,000

Year	Description	Type of Improvement	Current Cost Basis[1]	Future Cost Basis (3%)[2]	Future Cost Basis (5%)[3]
2033	Mid-Term Automatic Flushing Hydrants for Chlorine Residual	Quality	$69,000	$125,000	$184,000
-	**Total Mid-Term Projects**	-	**$137,611,000**	**$241,174,000**	**$348,952,000**
Average Cost Per Year				**$16,078,000**	**$23,263,000**

Notes:
1. *Engineering and Contingency estimates are included in each item at a value of Contingency 30% and Engineering 20% of the item cost.*
2. *Inflated to projected year dollars at 3% per year inflation rate.*
3. *Inflated to projected year dollars at 5% per year inflation rate.*

4.3 Long-Term Improvements

The recommended long-term capital improvements are summarized in Table 4-4. The long-term improvements include the following:

- Add 20.2 MGD Pump at West Vine Street Pumping Station to High Service Level.
- Transmission Main Rehabilitation or Replacement.
- East Supply Transmission Main to Vine Street Reservoir.
- Booster Pumping Station at I-80 Decommissioning / Demolition.
- Southwest Reservoir (4 MG above-grade) and Pipeline for Belmont Service Level.
- Saltillo Road Reservoir (3 MG above-grade) and Pipeline for High Service Level.
- Adams Street Reservoir II (5 MG above-grade) and Pipeline for High Service Level.
- 5.0 MGD Firm (8.0 MGD Installed) South Belmont Pumping Station to Belmont Service Level.
- Rokeby Reservoir (5 MG above-grade) and Pipeline for Southeast Service Level.
- Northwest Reservoir II (3 MG elevated) and Pipeline for Northwest Service Level.
- Long-Term Distribution System Extensions.

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Table 4-4 Recommended Long-Term Transmission and Distribution Systems Capital Improvements

Year	Description	Type of Improvement	Current Cost Basis[1]	Future Cost Basis (3%)[2]	Future Cost Basis (5%)[3]
Pumping, Storage and Transmission					
2041	Add 20.2 MGD Pump at West Vine Street Pumping Station to High SL	Pumping	$1,452,000	$3,323,000	$5,693,000
2042	Transmission Main Rehabilitation / Replacement	Transmission	$59,693,000	$140,671,000	$245,705,000
2044	East Supply Transmission Main to Vine St Reservoir	Transmission	$2,816,000	$7,041,000	$12,780,000
2045	Booster Pumping Station at I-80 Decommissioning/Demolition	Pumping	$216,000	$557,000	$1,030,000
2046	Southwest Reservoir and Pipeline for Belmont SL (4 MG above-grade)	Storage	$4,506,000	$11,952,000	$22,545,000
2048	Saltillo Road Reservoir and Pipeline for High SL (3 MG above-grade)	Storage	$5,167,000	$14,540,000	$28,502,000
2050	Adams Street Reservoir II and Pipeline for High SL (5 MG above-grade)	Storage	$5,529,000	$16,506,000	$33,625,000
2052	5.0 MGD Firm (8.0 MGD Installed) South Belmont Pumping Station to Belmont SL	Pumping	$1,618,000	$5,125,000	$10,849,000
2054	Rokeby Reservoir and Pipeline for Southeast SL (5 MG above-grade)	Storage	$7,075,000	$23,772,000	$52,299,000
2058	Northwest Reservoir II and Pipeline for Northwest SL (3 MG elevated)	Storage	$4,284,000	$16,201,000	$38,492,000
Distribution					

Year	Description	Type of Improvement	Current Cost Basis[1]	Future Cost Basis (3%)[2]	Future Cost Basis (5%)[3]
2050	Long-Term Distribution System Extensions	Distribution	$70,710,000	$211,086,000	$430,017,000
-	**Total Long-Term Projects**	-	**$163,066,000**	**$450,774,000**	**$881,537,000**
Average Cost Per Year				**$22,539,000**	**$44,077,000**

Notes:

1. Engineering and Contingency estimates are included in each item at a value of Contingency 30% and Engineering 20% of the item cost.

2. Inflated to projected year dollars at 3% per year inflation rate.

3. Inflated to projected year dollars at 5% per year inflation rate.

5.0 Water Main Replacement Program

The LWS distribution system consists of a wide range of pipe sizes, ages, and materials. As of the end of 2012, there were approximately 1,200 miles of water main ranging in size from 4-inch to 60-inch.

Currently, LWS has budgeted $4.0 million for main replacements in fiscal year 2013. This will replace approximately 5 miles, or 0.4 percent, of the overall distribution system.

LWS uses an asset ranking form to prioritize potential projects based on several criteria, including:

- Level of service consequence
- Damage consequence
- Water main condition and failure risk

The score from this asset ranking is combined with other factors such as:

- Break history
- Capacity improvements
- Fire flow improvements
- Opportunity projects (replacing water mains coincident with roadway projects)

It is recommended that LWS consider the condition assessment score with the consequence of failure to appropriately prioritize investments. Where the consequences of failure are high, direct assessment methods should be used periodically to determine pipeline conditions and take preventative actions as appropriate.

5.1 Valve and Hydraulic Inspection

LWS currently has a valve and hydrant inspection program. Hydrant valves are located and documented, but are not exercised since they do not affect customer service or fire protection provided. It is recommended to exercise the hydrant valves during this inspection. Inspecting the large diameter valves more frequently or annually is recommended.

5.2 Pipe Renewal

Through its current main renewal program, the City has kept its system-wide break rate below 20, which would be considered moderate and sustainable. This has been accomplished by replacing mains with significant numbers of breaks using conventional open-trench methods.

HDR Engineering, Inc. (HDR) recommends an increase in the length of water main replaced annually as a part of the LWS main replacement program. An annual replacement of 7 miles is recommended for LWS. To support this level of replacement approximately $6.3 million (in 2014 dollars) is required for a sustainable level of investment for the water main replacement program. This estimated cost is based on the assumption that the replacement projects will cost approximately the same (on a linear foot basis) as the replacement projects that occurred in 2013 (plus inflation).

6.0 Asset Management Water

A robust asset management system is recommended to provide LWS the information and tools necessary to make critical decisions for the water system. These decisions include maintenance scheduling and proactive prioritization of capital renewal and replacement projects.

LWS should develop defined and consistent business processes throughout all groups within the division. This includes consistent use of Geographic Information Systems (GIS) and CMMS, establishment of an asset management hierarchy, and routine syncing of GIS and the CMMS.

Another critical element of a robust asset management system is the implementation of a condition assessment process. This process will allow LWS to further extend the useful life of assets, reduce the potential of failure, and identify those assets that have the highest potential and consequence of failure in the system.

7.0 Financial Assessment

The 2013 Facilities Master Plan (Master Plan) includes a number of large and fairly significant capital projects. The financial assessment has demonstrated a number of items in which certain conclusions can be reached. These including the following:

- The City will need to issue additional long-term debt to support the Facilities Master Plan capital projects. If the City desires to issue additional debt, LWS's rates will need to be adjusted to meet debt service coverage requirements.
- The majority of the funding for the Facilities Master Plan capital projects will be funded on a "pay-as-you-go" basis using rate revenues. This will require the LWS to continually increase the level of funding of the CIP from rates over the period of the Master Plan from the current level.

The magnitude of the Missouri River Project raises a number of serious financial questions. Most importantly is whether this project is "affordable" and, if so, whether there is a financial strategy that LWS should consider for this particular project. There are no simple strategies to fund a project of this magnitude. However, it has concluded that LWS should consider the following strategy:

- As soon as possible, LWS should begin to set aside funds in a dedicated Missouri River Project reserve. The intent of this reserve is to begin to pre-fund the project such that it does not require 100 percent debt financing in 2040 or when built.
- The intent of funding this reserve is two-fold. First, it sets aside funds for the project, but more importantly, it begins to ramp up LWS's rates to a point at which LWS can support the eventual debt service payments associated with the project. Once the project is built, the financial strategy is that LWS will have gradually built into its rates, over the last 30 years, an amount that will pay a substantial portion of the annual debt service payment going forward. The key to this strategy is that it should minimize the need for a major rate adjustment (for example, a doubling of rates in a single year) at the time the debt is issued.
- A significant amount of funds will need to be collected annually and set aside in this dedicated reserve. Even with these funds set aside, LWS may be able to fund only 10 to 15 percent of the total expected project costs from this reserve.
- When the Missouri River Project is being built, LWS should deplete the dedicated reserve and apply those reserves against the project. The balance of any needed funds to construct the project will be obtained from the issuance of long-term debt.

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