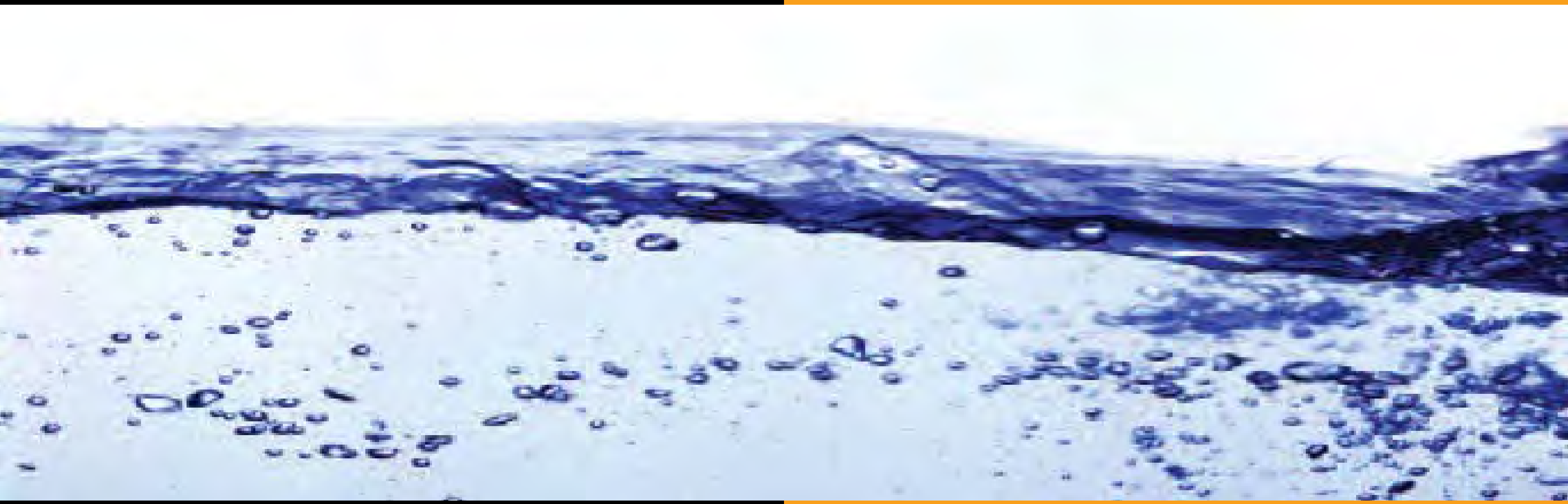
BUILDING A WORLD OF DIFFERENCE®



2007 FACILITIES MASTER PLAN UPDATE

Lincoln Water System

CITY OF LINCOLN, NE

Black & Veatch Project No. 148582

December 2009

BLACK & VEATCH
Building a world of difference®

OLSSON ASSOCIATES

BLACK & VEATCH
Building a world of difference.

Lincoln, Nebraska
Lincoln Water System
2007 Facilities Master Plan Update
City Project No. 701176

B&V Project 148582.0105
B&V File B-1.1
January 6, 2010

Nick McElvain
Operations Support Manager
Lincoln Water System
2021 North 27th Street
Lincoln, Nebraska 68503-1025

Dear Mr. McElvain:

Black & Veatch is pleased to deliver herewith the 2007 Facilities Master Plan Update. This report builds upon the 2002 Facilities Master Plan with a focus on extension of the distribution system through the Year 2057. It will also serve as a valuable document as you draft your capital improvements projects since it identifies the major improvement necessary in the immediate future.

As with previous master planning efforts, we would like to applaud the Lincoln Water System and other City entities on their concerted efforts to supply information, review interim documents, and provide valuable input at project meetings. These efforts have helped create a thorough report which serves as not only a roadmap for the utility, but also documentation of the past.

We are thankful to have once again served LWS on such an important project and look forward to continuing opportunities to assist you in the implementation of the recommended improvement.

Very truly yours,

BLACK & VEATCH CORPORATION

Andrew J. Hansen

Andrew J. Hansen
Project Manager

Enclosure

Acknowledgements

Black & Veatch gratefully acknowledges the cooperation and assistance of many people in the preparation of the Facilities Master Plan Update. The names of the people who participated and who were instrumental in the development and preparation of the report are presented below.

Lincoln Water System

Jerry Obrist.. Chief Engineer
Nick McElvain.. Water Operations Support Manager
Steve Owen.. Water Distribution Manager
John Miriovsky.. Water Production Manager
Arnie Radloff.. Utility GIS Specialist
Eric Lee.. Assistant Superintendent of Production-Operations

Management Team

Greg MacLean.. Director of Public Works & Utilities
Steve Masters.. Public Utilities Administration
Margaret Remmenga.. Public Works & Utilities Business Manager
Marvin Krout.. Director of Planning Department
Roger Figard.. City Engineer

Other City Departments

Sara Hartzell.. City of Lincoln - Lancaster County Planning Department
Ed Kouma.. City of Lincoln Watershed Management
Gary Thalken.. Lincoln Wastewater System

Black & Veatch

Chad Hill.. Project Director
Andrew Hansen.. Project Manager
Jerry Edwards.. Senior Technical Advisor
James Maher.. Project Engineer
Aaron Robinson.. Engineering Manager
Brian Lendt.. GIS Analyst

Olsson Associates

Shayne Huxoll.. Project Engineer, Main Replacement Program
John Olsson.. Principal

Table of Contents

Page No.

Table of Contents

Page No.

Table of Contents

Page No.

Table of Contents

Page No.

List of Tables

Page No.

List of Tables

List of Figures

List of Appendices

Executive Summary

Purpose and Study Area

This report has been prepared to provide the City of Lincoln with a guide for short-term and long-term improvements to the infrastructure for the Lincoln Water System. The recommended improvements plan presented herein will serve as a basis for the design, construction, and financing of facilities to meet the City's anticipated population growth and commercial development.

The Study Area for this investigation and report is shown on Figure ES-1. The various components of the Study Area have been delineated by the Lincoln-Lancaster County Planning Department. These components are described below:

- Existing City Limits: City limits of the City of Lincoln as of November 2005.
- Future Service Limits: The anticipated maximum extent areas to be served by utilities of the City of Lincoln by year 2032. This area is further divided into priority areas as follows:
  - Tier I – Priority A: Future service area that may be served by utilities by 2012.
  - Tier I – Priority B: The area for development beyond Priority A that may be served by utilities by 2019.
  - Tier I – Priority C: The phase of development areas to be served after Priority A and B that may be served by utilities by 2030.
- 50-year Long-term Potential Service Area: This report uses the limits of the Tier II to identify a long-term plan of improvements to provide service to year 2057.
- Beyond 50-year Service Area. Development into the Tier III area is area is beyond the 50-year time frame considered for this report.

Population

Historical population data for the City of Lincoln was obtained from the U.S. Census Bureau. The Lincoln-Lancaster County Planning Department provided aggregate population projections for the City of Lincoln for 5-year intervals from year 2010 to year 2050. The population at year 2057 was calculated from the application of a growth rate of 1.5 percent per year beyond the 2050 projections provided by the Lincoln-Lancaster County Planning Department.

Figure ES-1: Study Area

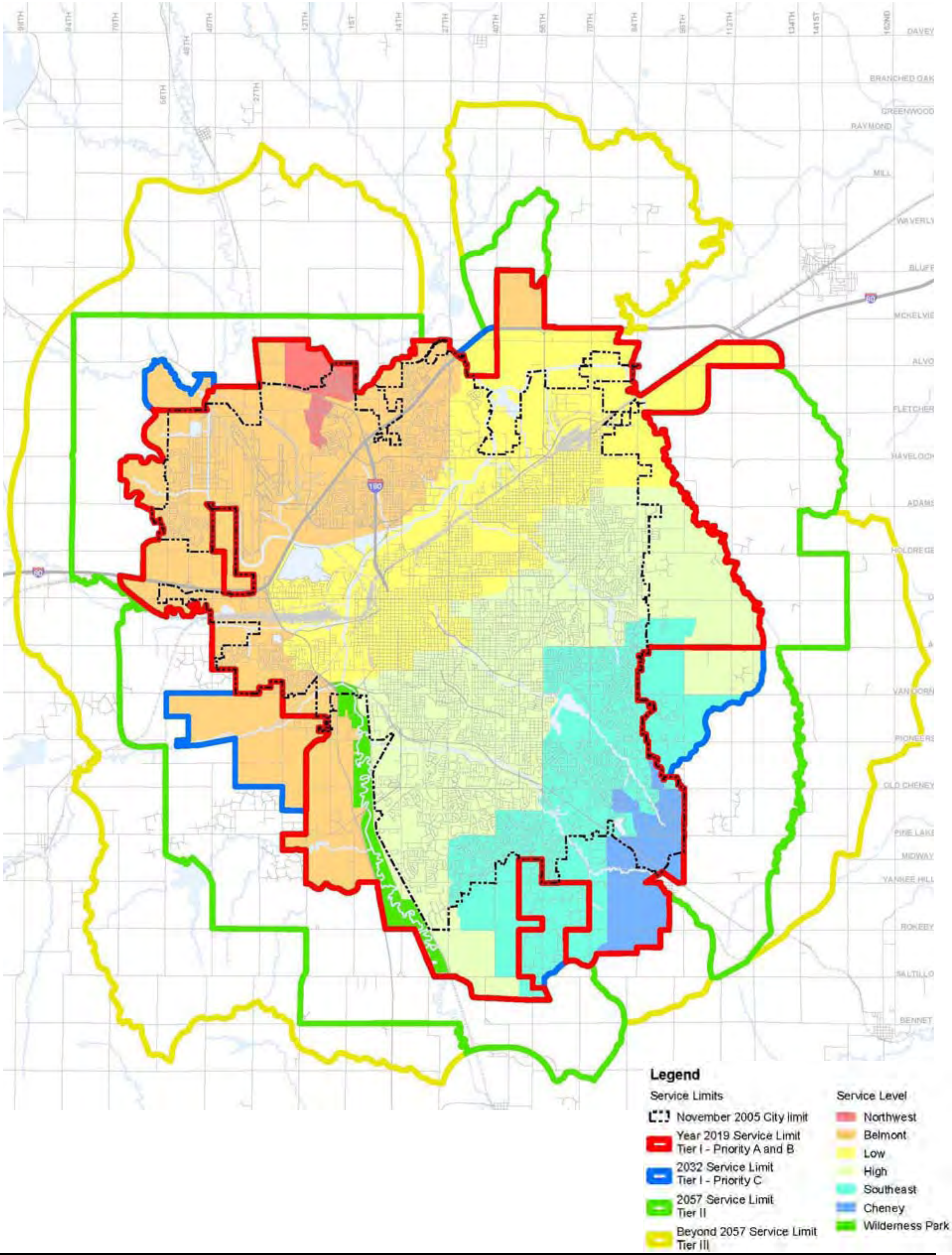
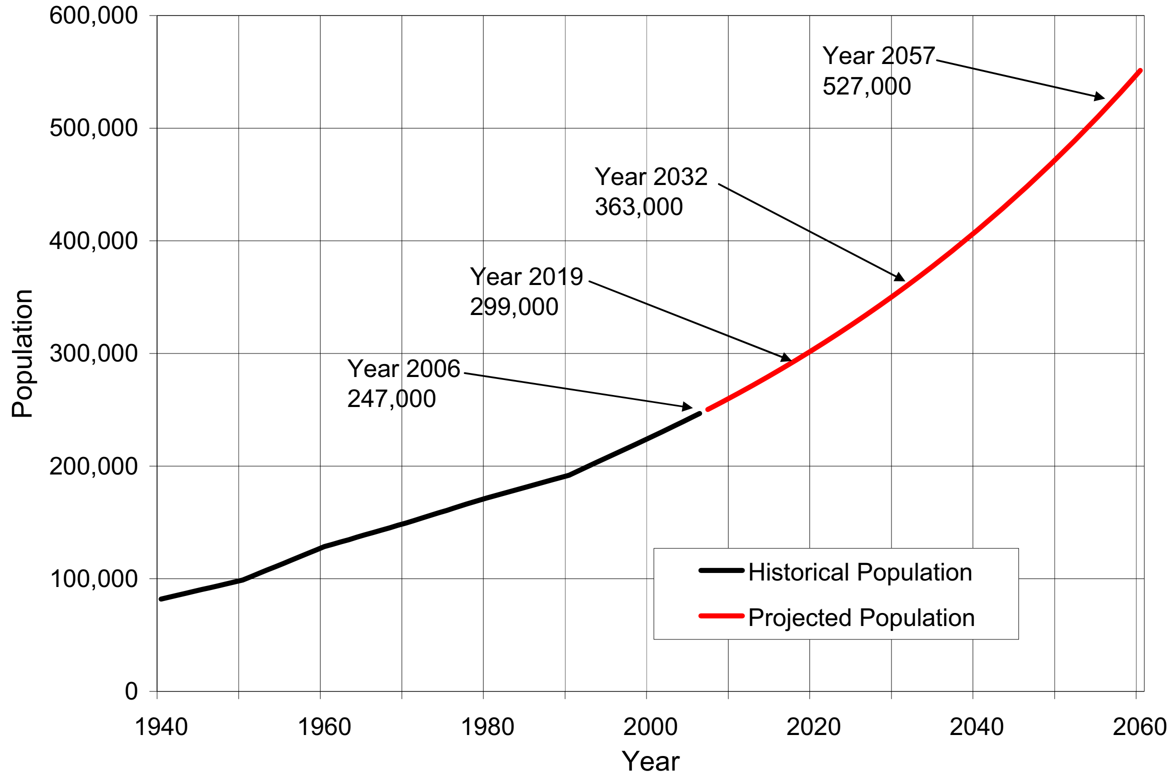


Figure ES-2: City of Lincoln Historical and Projected Population



The Lincoln-Lancaster County Planning Department provided a population projection for 2030 and spatial distributions of households for year 2030. Based on this information, projected populations for each service level was calculated as shown in Table ES-1.

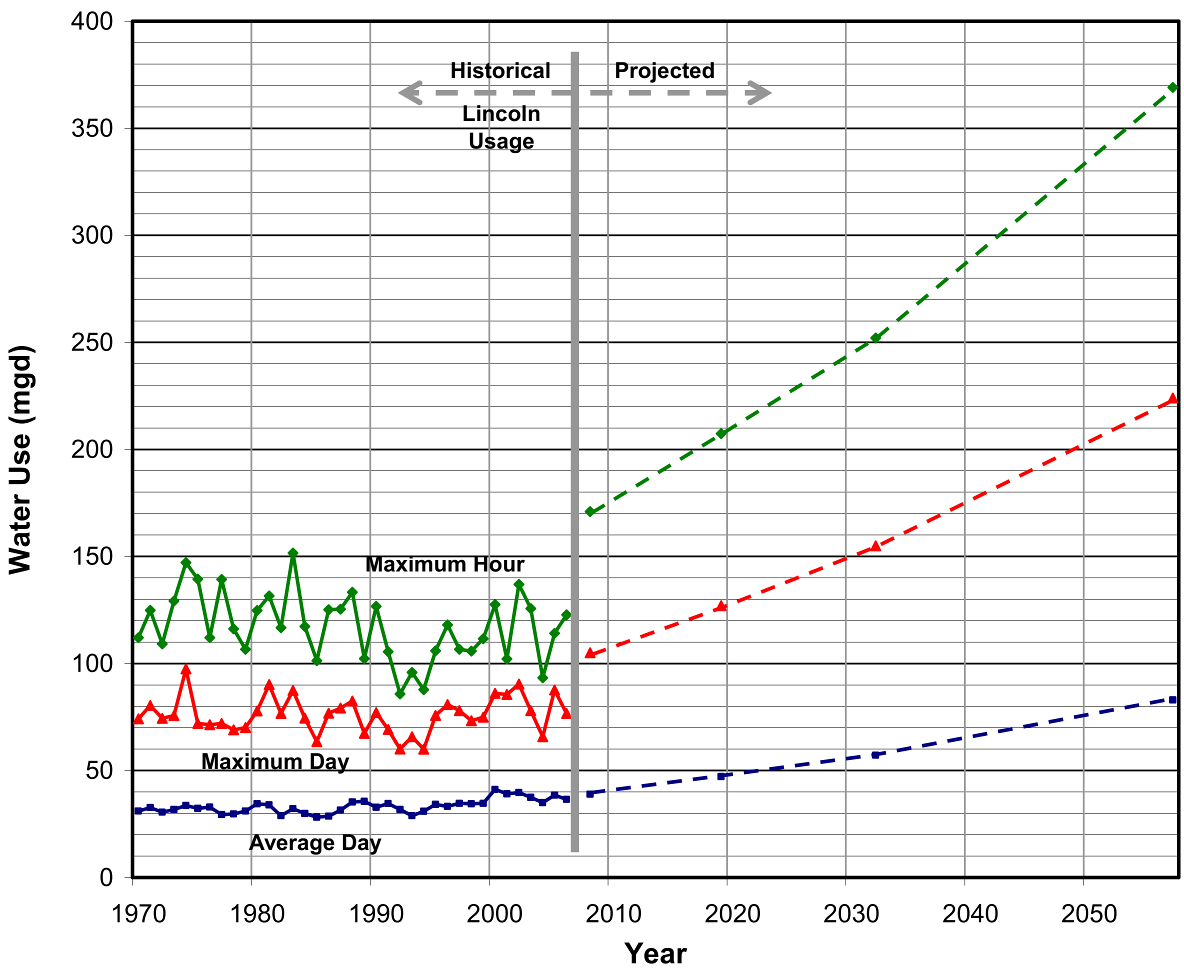
Table ES-1
Existing and Projected Population by Service Level

Service Level	Existing 2006	Short Term 2019	Mid Term 2032	Long Term 2057
Belmont	34,609	53,608	77,633	149,429
Low	76,668	81,726	86,884	100,612
High	100,908	112,275	127,863	165,003
Southeast	30,377	42,762	56,879	85,960
Northwest	1,765	3,632	5,487	9,762
Cheney	2,372	5,330	8,516	16,304
Total	**246,699**	**299,333**	**363,262**	**527,070**

Water Demands

Water demand projections were determined for the base year (year 2006) and years 2019 (short-term), 2032 (mid-term), and 2057 (long-term). The base year demand represents a normalized value for existing conditions which is generated based on review of multiple years of record. Therefore, the base year value may or may not be equal to actual values from year 2006, but represents an average condition upon which demands can be predicted. The theoretical demand could have occurred in year 2006 if the same criteria as for the projected water requirements were applied. Historical and projected total system water demands are shown on Figure ES-3

Figure ES-3: Historical and Projected Water Requirements



Design water demands used in the report for evaluation of recommended improvements are summarized in Table ES-2.

Table ES-2
Projected Water Requirements

Year	Average Day (mgd)	Maximum Day (mgd)	Maximum Hour (mgd)
Base Year	38.8	104.8	170.8
2019	47.2	127.3	207.4
2032	57.3	154.7	252.0
2057	83.0	223.9	369.0

Hydraulic Model

A distribution system hydraulic model was used to evaluate the ability of the system to meet existing and projected demand including fire flows, and to simulate aging and movement of the water in the distribution system to evaluate existing and potential water quality issues. The distribution system hydraulic model originally created for the *2002 Facilities Master Plan* was updated and calibrated to existing conditions. New pipes and facilities not contained in the 2002 model were added, and pipes that were identified as abandoned were removed to create the 2007 model. New demands based on recent metered sales were allocated to the 2007 model.

The 2007 model was calibrated to an actual condition that occurred over a 24-hour period on July 19, 2006. The calibration process consisted of simulating actual demands and operational controls, and verification of model connectivity and facility data until the model closely reproduced the recorded conditions. The calibrated 2007 model was used for subsequent hydraulic capacity and water quality evaluations conducted for this report.

Long-Range Plan

A long-range plan was developed based on providing water service to the year 2057 service area. The year 2057 service area includes development into the Tier II development limits as identified in the recent *2030 Comprehensive Plan* prepared by the Lincoln and Lancaster County Planning Department. Future boosted service levels were delineated along with the future boundaries of existing service levels. Hydraulic analyses using the 2007 Model were used in conjunction with previous evaluations as conducted for the *2002 Facilities Master Plan* to develop a long-range plan.

A significant component of the long-range plan is a new transmission main loop around the northwest portion of the service area to serve a much expanded Belmont Service Level. The northwest transmission main loop will meet projected demands and provide for possible expansion beyond the year 2057 (Tier II) limits. It will also maximize the effectiveness of existing facilities that currently provide service to the Belmont Service Level.

Significant transmission mains are also required on the east side of the distribution system. Most of the future east side mains originate from the Northeast Reservoir site, while others maximize and expand on existing pipeline, pumping, and storage capacities in the High Service Level and the Southeast Service Level.

Significant additional pumping will be required from the Northeast Reservoir and Pumping Station site. A new pumping station should be constructed for pumping to both the Belmont and High Service Levels from this location. This new pumping station will not be required until after year 2032.

Additional pumping capacity increases are also required at the Vine Street, A Street, and Pioneers pumping stations at various phases to accommodate growth. Several new storage facilities are recommended in the long-range plan with some of the storage required at various phases to accommodate growth.

The projected year 2057 maximum day demand of 224 mgd slightly exceeds the planned ultimate capacity of supply, pumping, and treatment facilities of existing sources; and the planned ultimate transmission capacity from the existing sources to Lincoln. Additional supply, treatment, pumping, and transmission capacity will be required to deliver the ultimate treated water capacity of 210 mgd to the City of Lincoln.

Future treated water to meet a demand in excess of 210 mgd is expected to be delivered to Lincoln in the southeast portion of the city. This future supply was considered in the long-range plan and the other phased improvements developed for this report.

Hydraulic Analyses and Development of Phased Improvements

A series of analyses were conducted using the hydraulic model, to identify recommended improvements required to resolve current deficiencies, meet projected demands, and improve water quality as described below:

- Base year analyses were conducted to evaluate the performance of the existing system under current peak demand conditions. The analyses indicated that the existing system can adequately meet current peak demand conditions, except that increasing demands are beginning to result in low pressures on the south portion of the system under maximum hour conditions.

- Year 2019 analyses were conducted to determine improvements required to serve the year 2019 service limits, and Year 2032 analyses were conducted to determine improvements required to serve the year 2032 service limits. These analyses provide the basis for the recommended improvements to increase transmission/distribution, pumping, and storage capacities.
- Fire flow analyses were conducted to determine fire flow capacity throughout the service area. Only a limited number of areas of deficient fire flows were identified for the existing system, and these deficient areas were considered in the development of immediate improvements recommendations.
- Analyses were performed to determine the performance of the distribution system during various proposed emergency conditions (Natural Disaster Preparedness Study). The findings of these analyses were reviewed and incorporated into the planning efforts.
- Water quality analyses were conducted to evaluate potential capital improvements and operational modifications to reduce water age and increase water quality in the distribution system. Only one capital improvement project (control valve at Pioneers Pumping Station) was identified as a result of the water age analyses. The water quality analyses will provide LWS with additional insight into how various demand and operational controls impact the quality of water in the distribution system.

Recommended Phased Improvements

A recommended phased improvements program was prepared to identify improvements for additional capacity and reliability through year 2032.

The "Phase I – Immediate Improvements" have been identified as a higher priority as a result of their immediate need or as a result of known or currently anticipated development. Phase I improvements also include improvements to correct fire flow deficiencies.

Improvements recommended to meet year 2019 demand conditions are referred to as "Phase II – 12-year Short-term Improvements". The Phase II improvements will extend service to the limits of the Tier I –Priority B area.

Improvements recommended to meet year 2032 demand conditions are referred to as "Phase III – 25-year Mid-term Improvements". The Phase III improvements will extend service to the limits of the Tier I – Priority C area.

Improvements recommended to provide service beyond the Tier I limits out to the Tier II limits are referred to as "Phase IV – 50-year Long-term Improvements".

Opinions of probable project costs were developed for the phased improvements program for all recommended mains, pumping, storage, and control facilities. While recommended improvements were developed to provide service to the limits of the Tier II area, this area is not planned to develop until after 2032 and detailed probable costs were not developed for these improvements. The recommended phased improvements through year 2032 are summarized in Table ES-3.

Table ES-3
Summary Recommended Improvements

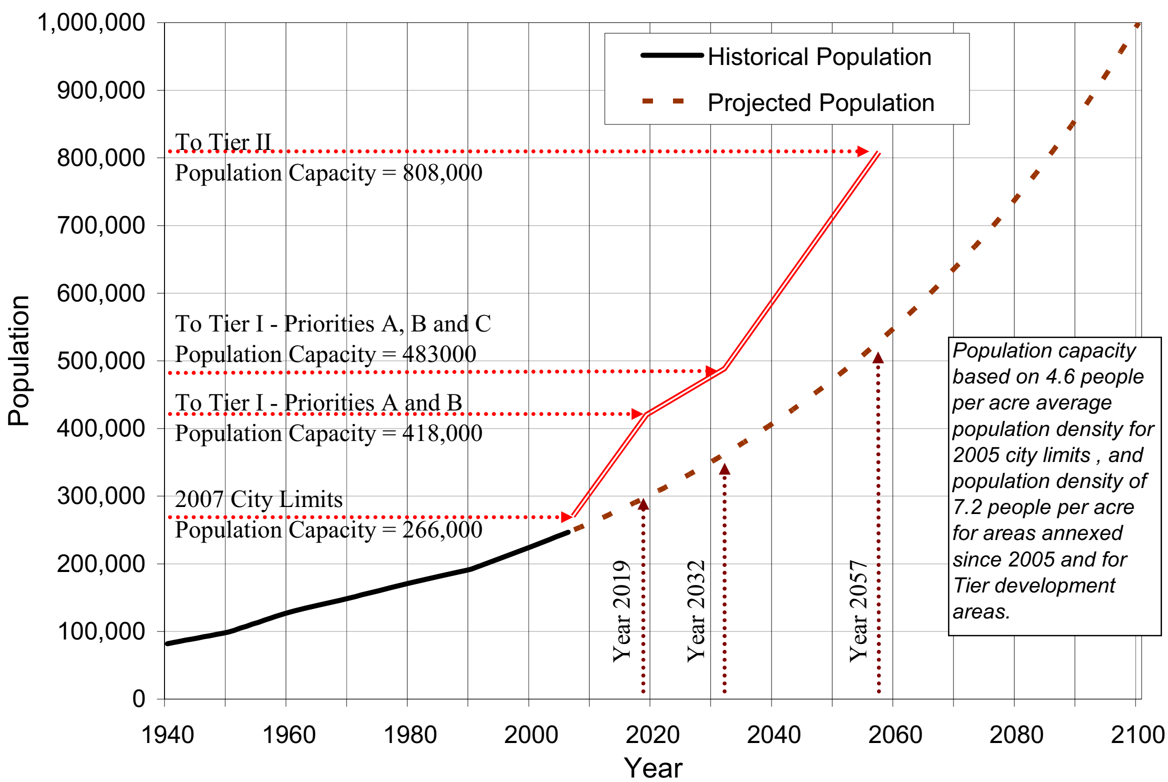
Description	Project Cost by Phase: Phase I Immediate Improvements	Phase II Short-term By Year 2019	Phase III Mid-term By Year 2032
Fire Flow Improvements (see table 8-1)	$460,000		
3.6 mgd Booster Pumping Station at I-80 west of N 56th St(3)	$1,290,000		
Control Valve in Pioneers Pumping Station	$50,000		
All Phase I Main Extensions	$11,010,000		
New 20 mgd pump at Vine Street PS to Southeast SL		$1,000,000	
New 10 mgd A St. Satellite Pumping Station to Low SL		$2,300,000	
New 20 mgd WTP High Service Pump No. 13		$1,000,000	
Pressure Monitoring Stations		$100,000	
All Phase II Main Extensions		$49,800,000	
Replace Pump SE1 at Vine St Southeast Pumping Station with 20 mgd Pump			$1,000,000
Add 5 mgd Pump No. 4 at Pioneers Pumping Station			$200,000
Replace Pump No. 10 at WTP with 20 mgd Pump			$1,000,000
Construct New High Service Pumping Station and add 20 mgd Pump No. 14 (include space for three units)			$4,600,000
8.0 mgd Yankee Hill Pumping Station(3)			$1,840,000
Additional Northeast Storage Capacity (10 MG buried below-grade)			$15,000,000
Saltillo Road Reservoir for High SL (4 MG above-grade)			$4,000,000
Southwest Reservoir for Belmont SL (5 MG above-grade)			$5,000,000
Northwest Reservoir for Northwest SL (1 MG elevated)			$2,000,000
All Phase III Main Extensions			$31,600,000
Total by Phase	**$12,810,000**	**$54,200,000**	**$51,390,000**

(1) Annual main replacement and other facility rehabilitation projects are not included in this table.
(2) All project costs are reported in year 2007 dollars, and include a 20 percent contingency, plus a 20 percent allowance for engineering, legal, and administrative costs.
(3) Reported pumping station capacities are firm capacity recommendation.

Annual Investment for Main Extensions

The total area annexed since 2005 plus the Tier I – Priorities A and B areas collectively provide over three times the minimum area required to support projected growth through year 2019. Figure ES-4 compares the population capacity for the tier areas to the projected population for the design years used in this report. It shows that more land will be made available for development in the coming years than is required to support the projected population. This excess of land provides for flexibility in the location of development, but may also commit the LWS to construction of transmission system improvements for potentially scattered development.

Figure ES-4: Projected Population and Tier Population Capacities



About $61 million of main extensions are recommended by year 2019 to serve the entire Tier I – Priority A and B areas. Only an additional $32 million of main extensions are then recommended by 2032 to serve the entire Tier I – Priority C area. On average, about $2.4 million is required for main extensions for every square mile of newly developed Tier I area.

At projected maximum densities, about 0.9 square mile of land per year would support the current growth rate. At densities more consistent with the existing City, about 1.4 square miles would be required. If development occurs adjacent to existing water utilities, improvements costs would be minimized and about $2.2 million to $3.4 million per year would be required to construct main extensions. However, if scattered development occurs, and it is necessary to construct transmission main improvements to serve the entire Tier I – Priority A and B areas by 2019, the annual CIP cost requirement will be closer to $5.1 million per year. The annual cost would decline after 2019 to about $2.5 million per year to serve the entire Tier I – Priority B area by 2032.

Water Main Replacement Program

The existing 6-year capital improvement plan for LWS includes $2.75 million for main replacement and rehabilitation. Assuming a 100 year service life for water mains, one percent of the system should be renewed every year to prevent the system from deteriorating. This level of funding translates into $6.92 million per year, which is approximately 2.5 times the current budget.

There is a need for an increase in the pipe replacement program budget in order to preserve the distribution system asset value. However, such a need must be assessed in the broader context of other priorities. Consideration should be given to developing a comprehensive "asset management plan" to establish future fiscal needs for preservation of LWS assets.

There is no formal replacement criteria established to identify which mains should be replaced. Consideration should be given to conducting a more detailed pipeline replacement plan using a matrix rating system to prioritize mains for improvement.

Consideration should also be given to developing a pipeline inspection program for large diameter mains to assess their condition and conduct proactive maintenance if required, to reduce the risk of future catastrophic failures.

1.0 Introduction

1.1 Purpose

This report has been prepared to provide the City of Lincoln with a guide for short-term and long-term improvements to the infrastructure for the Lincoln Water System. The recommended improvements plan presented herein will serve as a basis for the design, construction, and financing of facilities to meet the City's anticipated population growth and commercial development. The purpose of the recommended improvements is to provide an adequate and dependable supply of water to existing and future customers.

1.2 Scope

The study period for this investigation is from year 2006 through the year 2057. Evaluation of water demands by class and service level, and computer hydraulic analyses, were conducted for base year 2006 and design years 2019 and 2032, and 2057.

The study area for this investigation and report is shown on Figure 1-1 located at the end of this chapter. The various components of the Study Area have been delineated by the Lincoln-Lancaster County Planning Department in the *2030 Comprehensive Plan* as originally adopted on November 16, 2006 and amended on November 5/6, 2007. These components are described below:

- Existing City Limits: City limits of the City of Lincoln as of November 2005.
- Future Service Limits: The anticipated maximum extent areas to be served by utilities of the City of Lincoln by year 2030. This area is further divided into priority areas. These priority areas have been related to the water infrastructure recommendations contained in this report as follows:
  - Tier I – Priority A: Future service area of approximately 20 square miles that may be served by utilities by 2012.
  - Tier I – Priority B: The next area for development beyond Priority A. This report provides recommendations for water distribution piping facilities to be in place by year 2019 to provide water service to Priority B.
  - Tier I – Priority C: The later phase of development areas intended to be served after Priority A and B. The *2030 Comprehensive Plan* indicates that given the

current growth rates and infrastructure financing, development is not expected to occur until in Tier I – Priority C after 2020 or 2025. This report provides recommendations for water distribution piping and to be in place by year 2032 to provide water service to Priority C.

- 50-year Long-term Potential Service Area: The next tier of development to occur following the Tier I areas is identified at Tier II. This report used the limits of the Tier II along with long-term population projections provided by the Lincoln-Lancaster County Planning Department to identify a long-term plan of improvements to provide service to year 2057.
- Beyond 50-year Service Area. The 2030 Comprehensive Plan identifies the Tier III phase of development which would potentially occur after Tier II. Development into this area is beyond the 50-year time frame considered for this report, but is shown on Figure 1-1.

The principal elements of this study include the following:

- Update the historical water use trends and projections of future water requirements as originally developed for the *2002 Facilities Master Plan*, based on recent population projections provided by the Lincoln-Lancaster County Planning Department.
- Evaluate the adequacy of existing distribution system components under present and future conditions.
- Update the computer model of the Lincoln water distribution system in H2OMAP hydraulic analysis software and provide the updated model to LWS for use by LWS staff.
- Perform hydraulic analyses including 1) capacity analysis of the distribution system to meet present and future water demands, 2) water age analyses to identify potential capital improvements and operational modifications to reduce water age and potentially improve water quality, and 3) fire flow analyses to identify potential areas with deficient fire flows and to develop improvements to correct the identified deficiencies.
- Review the current main replacement program, main break history, and distribution system maintenance schedules and identify recommendations to improve information collection, and to estimate pipeline life cycle costs.

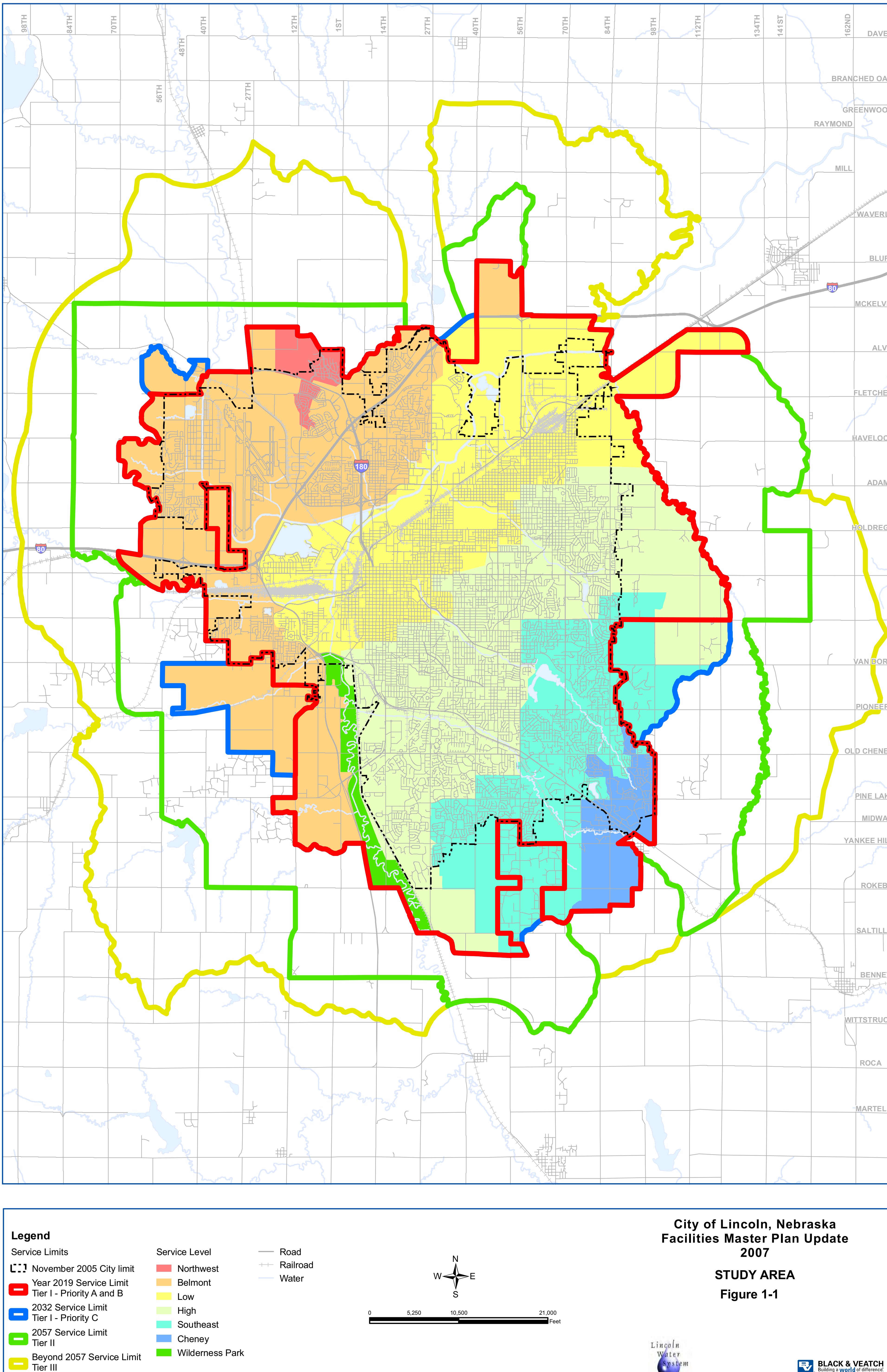
- Prepare an update of recommended water system improvements, including a phased construction program and opinions of probable cost. The distribution system improvements recommended in this report are staged to address existing system deficiencies and to coincide with anticipated development.

1.3 Abbreviations

Abbreviations used in this report are as follows:

AD	(Annual) Average Day
AM	Average Month
AWWA	American Water Works Association
BG	Billion Gallons
BPS	Booster Pumping Station
CCI	Construction Cost Index
CIP	Capital Improvements Program
D/DBPR	Disinfection/Disinfectant By-Product Rule
El.	Elevation
ENR	Engineering News Record
EPA	(United States) Environmental Protection Agency
EPS	Extended Period Simulation
ESRI	Environmental Systems Research Institute
ft.	Feet
gpcd	Gallons per capita per day
gpm	Gallons per minute
GIS	Geographic Information Systems
HG	Hydraulic Gradient
HGL	Hydraulic Grade Line
hp	Horsepower
ICI	Industrial/Commercial/Institutional
IDSE	Initial Distribution System Evaluation
in.	Inch
ISO	Insurance Services Office
LWS	Lincoln Water System
MD	Maximum Day

MG	Million Gallons
mgd	Million gallons per day
MH	Maximum Hour
MM	Maximum Month
NRW	Non-revenue Water
PRV	Pressure Reducing Valve
psi	Pounds per square inch
rpm	Revolutions per minute
SCADA	Supervisory Control And Data Acquisition
SL	Service Level
SMP	Standard Monitoring Plan (for Stage 2 D/DBPR)
SSS	System Specific Study (for Stage 2 D/DBPR)
TAZ	Traffic Analysis Zone
TDH	Total Dynamic Head
TTHM	Total Trihalomethanes
UNF	Unaccounted-for Water
USGS	United States Geological Survey
WSE	Water Surface Elevation
WTP	Water Treatment Plant



Data source: City of Lincoln, B&V
Path: C:\Projects\Lincoln\Lincoln_148582.0100\11GIS\01MapDocuments\Study area map.mxd Date Printed: February 17, 2008 Map By: B. Lendt

2.0 Population

Development of a comprehensive water system master plan begins with an evaluation of the area's historical population trends and projected growth patterns. To accurately predict future water demands, it is necessary to determine the magnitude, direction, and characteristics of future population growth.

2.1 Methodology

The study periods for this project include existing conditions for 2006, Tier I – Priority A and B at year 2019, and Tier I – Priority C at year 2032. In addition, consideration was given to long-term Tier II growth to year 2057 in the development of recommended improvements.

The LWS service population is limited to the area within the City Limits. Areas outside the City Limits are served by individual well supplies, or by Rural Water Districts who do not purchase water from the City.

2.2 City of Lincoln Population

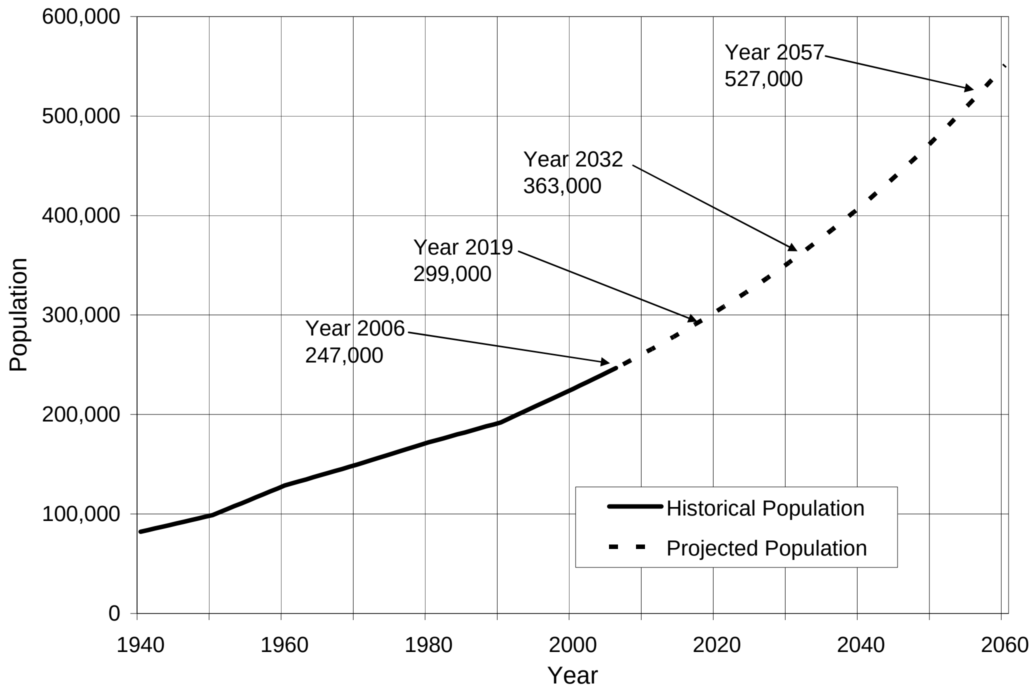
Historical population data for the City of Lincoln was obtained from the U.S. Census Bureau. The Lincoln-Lancaster County Planning Department provided aggregate population projections for the City of Lincoln for 5-year intervals from year 2010 to year 2050. The population at year 2057 was calculated from the application of a growth rate of 1.5 percent per year beyond the 2050 projections provided by the Lincoln-Lancaster County Planning Department. Historical and projected populations for the City of Lincoln are summarized in Table 2-1 and shown in Figure 2-1 located at the end of this chapter.

Table 2-1
City of Lincoln Population
(Historical and Projected)

Year	Population[3]	Average Annual Growth	
		Persons	%
1940[1]	81,984	--	--
1950[1]	98,884	1,690	1.9
1960[1]	128,521	2,964	2.7
1970[1]	149,518	2,100	1.5
1980[1]	171,932	2,241	1.4
1990[1]	191,972	2,004	1.1
2000[1]	225,581	3,361	1.6
2006 (Base Year) [2]	246,699	3,515	1.5
2010[4]	261,796	3,782	1.5
2019 (Short Term) [5]	299,334	4,171	1.5
2030[4]	352,601	4,842	1.5
2032 (Mid Term) [5]	363,258	5,329	1.5
2050[4]	474,903	6,203	1.5
2057 (Long Term) [6]	527,070	7,452	1.5

(1) U.S. Census Bureau.
(2) Calculated from TAZ population data provided by the Lincoln-Lancaster County Planning Department falling within the 2006 service area.
(3) Population projections include growth in annexation areas including existing population within annexation areas.
(4) Projections by Lincoln-Lancaster County Planning Department dated March 27, 2001.
(5) Population interpolations.
(6) Population extrapolation based on growth rate of 1.5 percent per year.

Figure 2-1: City of Lincoln Historical and Projected Population



2.3 Population Distribution

2.3.1 General

The Lincoln-Lancaster County Planning Department provided spatial distributions of households within the county for existing year 2006, and year 2030. The household count data was delineating by the Lincoln-Lancaster County Planning Department into total of 502 traffic analysis zones (TAZ) that covered the entire study area including the Tier III development limits. The data included average population-per-household and per-group-quarter for 2006. For this report, it was assumed that the future population-per-household and per-group-quarters would remain the same as 2006.

The population for each TAZ was calculated for years 2006 and 2030. The population for each TAZ for year 2032 was then calculated by allocating the aggregate population growth of 1.5 percent to the TAZ level, by extrapolating the growth by TAZ for year 2030. The population for each TAZ for year 2019 was calculated based on interpolation of year 2006 and year 2032 population by TAZ with slight adjustments as required to match the aggregate projected population. No TAZ data was provided beyond year 2030, so the population for each TAZ for year 2057 was forecasted by allocating the aggregate population increase only to TAZs that showed growth between 2006 and 2030.

2.3.2 Population by Service Level

Year 1980 and 1990 populations by service level were presented in the 1995 Master Plan and year 2000 populations were presented in the *2002 Facilities Master Plan*. Based on the population counts by TAZ, the population by service level was calculated for this study for year 2006. There are two new service levels developed by LWS since the 2000 census. The Cheney Service Level provides service toe high ground in the southeastern corner of the City. The Northwest Service Level provides service to high ground in the northwestern portion of the City.

For year 2006, the percentage of population served, and the percentage split to service level was estimated for each TAZ based on area coverage and location of developed areas. The 2006 service population was then calculated by multiplying the total population in the TAZ times the percent served. The results were tabulated, checked against the year 2000 Census Bureau population, and adjusted slightly to be consistent the census population. The population by service level was then tabulated as shown in Table 2-2.

Table 2-2
Historical Population by Service Level

Service Level	1980[(1)]	1990[(1)]	2000[(1)]	2006[(2)]
Belmont	14,500	18,890	31,830	34,609
Low	64,800	67,100	71,466	76,668
High	81,600	89,210	94,840	100,908
Southeast	12,350	16,770	27,445	30,377
Northwest	-	-	-	1,765
Cheney	-	-	-	2,372
Total	**173,250**	**191,970**	**225,581**	**246,699**

(1) From 2002 Facilities Master Plan
(2) Calculated for this study based on population by TAZ as provided by Lincoln-Lancaster County Planning Department

For year 2032, the service population was calculated as the projected population within the Future Service Limit for year 2030 of 352,600 with a 1.5 percent growth per year for two years. The population by service level was calculated similar to year 2006, except that the service population was slightly adjusted to match the projected year 2032 City of Lincoln population of 363,258.

The service population for year 2019 was calculated based upon straight line interpolation of service populations by TAZ for year 2006 and year 2032 with slight adjustments made to match the aggregate population projection of 299,333 for that study year.

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

Table 2-3
Existing and Projected Population by Service Level

Service Level	Base Year	Short Term 2019	Mid Term 2032	Long Term 2057
Belmont	34,609	53,608	77633	149,429
Low	76,668	81,726	86884	100,612
High	100,908	112,275	127863	165,003
Southeast	30,377	42,762	56879	85,960
Northwest	1,765	3,632	5487	9,762
Cheney	2,372	5,330	8516	16,304
Total	**246,699**	**299,333**	**363,262**	**527,070**

3.0 Water Requirements

3.1 General

A water utility must be able to supply water at rates that fluctuate over a wide range. Yearly, monthly, daily, and hourly variations in water use occur, with higher use during dry years and in hot months. Also, water use typically follows a diurnal pattern, being low at night and peaking in the early morning and late afternoon. Rates most important to the hydraulic design and operation of a water treatment plant and distribution system are average day (AD), maximum day (MD), and maximum hour (MH).

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

Maximum day use is the maximum quantity of water used on any one day of the year. The maximum day rate is used to size water supply hydraulics, treatment facilities, and pumping stations. The raw water facilities must be adequate to supply water at the maximum day rate, and the treatment facilities must be capable of processing this quantity of water.

Maximum hour use is the peak rate at which water is required during any one hour of the year. Since minimum distribution system pressures are usually experienced during maximum hour, the sizes and locations of distribution facilities are generally determined on the basis of this condition. Maximum hour water requirements are partially met through the use of strategically located system storage. The use of system storage minimizes the required capacity of transmission mains and permits a more uniform and economical operation of the water supply, treatment, and pumping facilities.

3.2 Historical Water Production and Usage

3.2.1 Total System

Historical water production was summarized by the City and provided in electronic format and also reviewed from the *2002 Facilities Master Plan*. Monthly Water Treatment Plant Operating Reports (Monthly Reports) were also provided for the fiscal year of 2006. The Monthly Reports include information on pumping from the well fields, finished water pumping at the treatment plant, and distribution system usage (Lincoln Usage). Historical usage for the 12 year period between 1994 and 2006 are summarized in Table 3-1.

The Lincoln Usage reported is the City's calculation of the distribution system usage after transmission. This value is calculated in the City's report by summing the pumping from transmission facilities which deliver water to the distribution system (Northeast Pumping Station, 51st Street Pumping Station, Merrill Street Pumping Station, and "A" Street Pumping Stations).

Table 3-1
Historical Water Usage

Year	Total Annual Pumpage, BG (1)	Lincoln Usage, BG (2)	AAD Demand, (mgd)	Maximum Day Demand, (mgd) (3)	Maximum Hour Usage, (mgd) (4)	MD:AD	MH:AD	MH:MD
1994	11.3	11.3	30.9	59.9	87.8	1.9	2.8	1.5
1995	12.5	12.5	34.2	75.7	106.0	2.2	3.1	1.4
1996	12.1	12.1	33.2	80.8	118.3	2.4	3.6	1.5
1997	12.9	12.7	34.7	78.0	106.6	2.5	3.2	1.3
1998	12.6	12.6	34.5	78.4	105.8	2.3	2.9	1.3
1999	12.7	12.7	34.7	74.9	111.5	2.2	2.7	1.2
2000	15.0	15.0	41.2	86.0	127.5	2.0	3.1	1.5
2001	14.5	14.3	39.1	85.5	102.1	2.2	2.6	1.2
2002	14.6	14.5	39.7	90.4	136.9	2.3	3.5	1.5
2003	13.7	13.7	37.5	78.0	125.7	2.1	3.3	1.6
2004	12.8	12.8	35.0	65.8	93.3	1.9	2.7	1.4
2005	13.8	14.1	38.5	87.6	114.1	2.3	3.0	1.3
2006	14.0	13.3	36.5	76.7	122.8	2.1	3.0	1.5
Average	**13.3**	**13.2**	**36.1**	**78.7**	**109.8**	**2.2**	**3.0**	**1.4**

(1) From 'Past Water Demand Parameters 1973-2006 (a)' provided by LWS.
(2) Year 1994-2000 from *2002 Facilities Master Plan*, Year 2001-2006 from Production data provided by LWS.
(3) From Production data provided by LWS, except for 1997 through 2000, and 2006 which were calculated from hourly SCADA data.
(4) Year 1994-2000 from 'Past Water Demand Parameters 1973-2006 (a)' , Year 2001-2006 from Production data provided by LWS except for 1997 through 2000, and 2006 which were calculated from hourly SCADA data.

3.2.2 By Service Level

Daily and hourly demands were calculated for each service level for this study for the year 2006 and appended to similar data calculated for the *2002 Facilities Master Plan*. Historical maximum day and maximum hour demands by service level are shown in Table 3-2 and 3-3, respectively.

Table 3-2
Historical Maximum Day Demands (mgd) by Service Level

Date	Belmont (1)	Low	High	Southeast (2)	Total
07/26/1997	8.64	17.19	41.55	10.43	77.97
07/20/1998	8.22	24.79	35.88	9.53	78.42
07/29/1999	8.57	19.54	35.89	10.91	74.92
06/07/2000	8.86	29.18	37.50	10.44	85.98
07/19/2006	13.73	18.43	33.57	10.96	76.69
Average	**9.49**	**22.22**	**36.28**	**9.88**	**77.92**
Maximum	**13.73**	**29.18**	**41.55**	**10.96**	**85.98**

(1) Includes the Northwest booster district demands at year 2006.
(2) Includes the Cheney booster district demands at year 2006.

Table 3-3
Historical Maximum Hour Demands (mgd) by Service Level

Date	Belmont (1)	Low	High	Southeast (2)
07/26/1997	11.68	23.34	59.46	15.72
07/20/1998	13.43	37.91	52.26	19.32
07/29/1999	12.52	26.80	52.98	20.28
06/07/2000	15.38	51.25	53.98	19.98
07/19/2006	21.96	38.02	54.09	27.28
Average	**14.99**	**35.46**	**54.55**	**20.51**
Maximum	**21.96**	**51.25**	**59.46**	**27.28**

(1) Includes the Northwest booster district demands at year 2006.
(2) Includes the Cheney booster district demands at year 2006.

Peaking factors and diurnal patterns were developed as part of the *2002 Facilities Master Plan* and have been reviewed for this study against those developed in the model calibration efforts and the design diurnal peaking factors from 2002 were used for this update. The peak demand diurnal curves consistently indicate that the peak hour demand occurs during the morning hours, with a second, lower peak hour demand in the evening hours.

3.3 Historical Metered Sales

3.3.1 Total System

Annual water sales were reviewed to determine the mix of residential and non-residential water use, and per capita water use rates. This information provides a basis for the breakdown and distribution of projected water demands.

Fiscal year metered sales for years 1986 through 2000 were provided in the *2002 Facilities Master Plan* and year 2001 through 2006 metered sales data was provided by LWS for this update. The "Summary of All Cycles" report provides metered sales summarized by Residential, Non-Residential, and High User categories.

Metered sales were compared to historical "Lincoln Usage" to evaluate the amount of non-revenue (NRW) water in the distribution system. As described previously in this report, "Lincoln Usage" as reported on the monthly reports does not include losses in transmission and the distribution system. Over the past 25 years, non-revenue water in the distribution system has averaged seven percent of the total average day "Lincoln Usage".

Metered sales by user class and non-revenue water (in the distribution system) for the 20 year period between 1986 and 2006 are summarized in Table 3-4.

Table 3-4
Historical Metered Sales

Fiscal Year[1][2][3]	Historical Metered Sales					AD	NRW
	Residential		Non-Residential		Total	(Lincoln Usage)	(Non-Revenue Water)
	(mgd)	(%)	(mgd)	(%)	(mgd)	(mgd)	(% of AD)
1987	[4]	[4]	[4]	[4]	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	6.0
1992	17.7	61	11.2	39	28.9	31.8	9.0
1993	16.0	60	10.5	40	26.5	28.9	8.4
1994	18.0	61	11.3	39	29.4	31.0	5.3
1995	20.1	63	11.9	37	32.0	34.2	6.6
1996	19.0	62	11.7	38	30.7	33.2	7.6
1997	20.2	62	12.5	38	32.7	34.7	5.6
1998	19.6	61	12.5	39	32.1	34.5	6.9
1999	19.3	61	12.4	39	31.7	34.7	8.7
2000	23.7	65	12.9	35	36.6	41.2	11.1
2001	21.8	63	12.7	37	34.5	39.1	11.8
2002	23.8	65	12.6	35	36.4	39.7	8.3
2003	23.1	65	12.3	35	35.4	37.5	5.6
2004	22.1	65	11.7	35	33.8	35	3.4
2005	24.5	67	12.2	33	36.7	38.5	4.7
2006	22.6	65	11.9	35	34.5	36.5	5.5
Average	**20.7**	**63**	**12.1**	**37**	**32.6**	**35.0**	**6.8**

(1) Fiscal year basis is from September through August. Example: FY 2000 is from September 1999 through August 2000.
(2) 1986 through 2000 data is from *2002 Facilities Master Plan* by Black & Veatch.
(3) 2001 through 2006 FY metered sales from bi-monthly metered sales report “Summary of All Cycles.”
(4) Data Not Available.

Metered sales data was related to historical population to provide an indication of historical per capita uses. For purposes of this evaluation, the population was assumed to grow in even increments between census years. Per-capita usage rates for the years 1986 through 2006 are summarized in Table 3-5.

Table 3-5
Historical Per-Capita Usage

Fiscal Year[1]	Population [2]	Residential Sales[3]		Total Sales[3]		AD (Lincoln Usage)	
		Total (mgd)	Per-Capita (gcd)	Total (mgd)	Per-Capita (gcd)	Total (mgd)	Per-Capita (gcd)
1986	183,956	(4)	(4)	27.5	149	28.7	156
1987	185,960	(4)	(4)	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	104	32.5	167	34.6	177
1992	198,694	17.7	89	28.9	146	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	89	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	182
2001	228,400	21.8	95	34.5	151	39.1	171
2002	231,700	23.8	103	36.4	157	39.7	171
2003	234,700	23.1	98	35.4	151	37.5	160
2004	236,100	22.1	94	33.8	143	35	148
2005	238,600	24.5	103	36.7	154	38.5	161
2006	239,200	22.6	94	34.5	144	36.5	153
Average	**212,052**	**20.7**	**96**	**32.4**	**153**	**34.7**	**164**

(1) Fiscal year basis is from September through August. Example: FY 2000 is from September 1999 through August 2000.
(2) 1990 and 2000 populations are from US Census Bureau and intermediate years are interpolated by straight-line. Year 2001 through 2006 population data obtained from 'Past Water Demand Parameters 1973-2006 (a)' provided by LWS.
(3) 1986 through 2000 data is from *2002 Facilities Master Plan* by Black & Veatch and 2001 through 2006 FY metered sales from bi-monthly metered sales report "Summary of All Cycles".
(4) Data not available.

The metered sales data in Table 3-4 demonstrate that the residential sales as a percentage of total sales has increased slightly over the past two decades. This increase in the residential sales as a percent of total sales is likely a result of a trend toward more service-oriented commerce with less industrial manufacturing. For projected water demands for this study, it is assumed that per-capita use will remain steady and that residential sales will continue to account for about 65 percent of the total sales.

3.3.2 Sales by Service Level

The City provided fiscal year 2006 geocoded metered sales for every account in the LWS distribution system consisting of approximately 79,000 records. The information included account number, account address, annual sales, and a user classification code. The account number identified the meter cycle for each account. Year 2006 metered sales were then summarized by service level as shown in Table 3-6.

Table 3-6
Year 2006 Metered Sales by Service Level [(1)]

Service Level	Fiscal Year 2006 Metered Sales (mgd)		
	Residential	**Non-Residential**	**Total**
Belmont	3.5	1.9	5.3
Low	5.1	6.7	11.8
High	9.3	3.3	12.6
Southeast	4.6	0.6	5.2
Northwest	0.3	0.2	0.5
Cheney	0.5	0.1	0.6
Total	**23.2**	**12.8**	**36.0**

(1) Geocoded metered sales values provided by LWS do not exactly match the metered sales calculated from bi-monthly metered sales report “Summary of All Cycles” for 2006.

The service population for each service level from Chapter 2 was reviewed against the metered sales by zone and the residential per-capita water use for each service level for year 2006 was calculated as shown in Table 3-7.

Table 3-7 Year 2006 Actual Per-capita Residential Use by Service Level			
	Fiscal Year 2006 Metered Sales (mgd)		
Service Level	**Population**	**Residential Sales (mgd)**	**Residential Per-capita Use (gpcd)**
Belmont	34,609	3.5	101
Low	76,668	5.1	67
High	100,908	9.3	92
Southeast	30,377	4.6	151
Northwest	1,765	0.3	170
Cheney	2,372	0.5	211
Total System	**246,699**	**23.3**	**94**

3.4 Water Use Projections

Water use projections were developed for the total system (in aggregate) for each design year. In addition, water demand projections by service level were determined for the base year (year 2006) and years 2019 (short-term) and 2032 (mid-term). The base year demand represents a normalized value which is generated based on review of multiple years of record. Therefore, the base year value may or may not be equal to actual values from year 2006, but represents an average condition upon which demands can be predicted. The theoretical demand would have occurred in year 2006 if the same criteria as for the projected water requirements were applied.

3.4.1 Total System

The residential per capita water use, percentage residential and ICI use, and non-revenue water were used to determine the base year 2006 and design years 2019 and 2032 water demands. Residential water use is based on a per capita basis while non-residential sales and non-revenue water use are determined on a proportional basis. For this study, residential use is considered to be water used by domestic customers in houses, apartments, and dormitories. Non-residential use includes water used by businesses, industries, hotels, hospitals, and similar establishments.

Design criteria for projection of water demands were developed in detail for the *2002 Facilities Master Plan*. Those design criteria were used for this 2007 Update with minor adjustments based on recent usage data. The residential per capita was increased slightly and the residential percentage of total sales was increased slightly. In addition, the percentage of non-revenue was reduced slightly. The resulting per capita total distribution usage was therefore reduced to 157 gcd from the previous value of 160 gcd. This reduction is consistent with the historical downward trend in per-capita usage. Additional reductions in per-capita usage resulting from a low usage futures are not expected to be significant. Future reductions in per-capita usage as a result of water conservation measures should be re-evaluated in the next master plan report.

Design values for projections of water requirements are summarized in Table 3-8 and the projected water requirements as a function of these criteria are shown in Table 3-9.

Table 3-8
Design Criteria for Projected Water Requirements

Per-capita Residential Metered Sales	96 gpcd
Residential Sales as Percent of Total Metered Sales	65%
Per Capita Total Metered Sales	148 gpcd
Non-revenue Water (Percent of Total Distribution Usage)	6.25%
Total Distribution Usage expressed as Per Capita Usage	157 gpcd
Transmission and Treatment Uses (% of Lincoln Usage)	3%
Transmission and Treatment Uses (per capita basis)	5 gpcd
MD/AD Peaking Factor	2.7
MH/AD Peaking Factor	4.4

Table 3-9
Projected Water Requirements (Total System)

Description	Base Year	Design Year 2019	Design Year 2032	Design Year 2057
Population	246,699	299,334	363,258	527,070
Residential Metered Sales (mgd)	23.7	28.7	34.9	50.6
Total Metered Sales (mgd)	36.4	44.2	53.7	77.8
Non-revenue Water (mgd)	2.3	2.8	3.4	4.9
AD Lincoln Usage (mgd)	38.8	47.1	57.3	83.0
MD Lincoln Usage (mgd)	105	127	155	224
MH Lincoln Usage (mgd)	171	207	252	365
AD Transmission and Treatment Uses (mgd)	1.2	1.5	1.8	2.6
AD Production (mgd)	40.0	48.6	59.0	85.5
MD Production (mgd)	108	131	159	231

Historical and projected total system water demands are shown on Figure 3-1 located at the end of this chapter.

3.4.2 Projections by Service Level

Based on the total system demands by class and historical uses, base year and future years 2019 and 2032 average day water requirements were determined as shown in

Table 3-10. The base year demands and projected design year 2019 and 2032 demands by service level were allocated and used for computer hydraulic analyses as described later in this report.

Table 3-10
Projected AD Water Demands by Class and Service Level

Service Level	Res. Per-capita (gpcd)	Res. Sales (mgd)	Res./Total Sales (%)	Non-Res. Sales (mgd)	Total Sales (mgd)	NRW (%)	NRW (mgd)	AD (mgd)
Base Year								
Belmont	80	2.8	63	1.6	4.4	6.25	0.3	4.7
Low	75	5.8	45	7.0	12.8	6.25	0.9	13.6
High	105	10.6	75	3.5	14.1	6.25	0.9	15.1
Southeast	133	4.0	90	0.4	4.5	6.25	0.3	4.8
Northwest	120	0.2	70	0.1	0.3	6.25	0.02	0.3
Cheney	132	0.3	95	0.0	0.3	6.25	0.02	0.4
Total	**96**	**23.7**	**65**	**12.7**	**36.4**	**6.25**	**2.4**	**38.8**
Design Year 2019								
Belmont	80	4.3	63	2.5	6.8	6.25	0.5	7.3
Low	75	6.1	43	8.1	14.3	6.25	1.0	15.2
High	103	11.6	74	4.1	15.6	6.25	1.0	16.7
Southeast	130	5.6	90	0.6	6.2	6.25	0.4	6.6
Northwest	120	0.4	70	0.2	0.6	6.25	0.04	0.7
Cheney	130	0.7	95	0.0	0.7	6.25	0.05	0.8
Total	**96**	**28.7**	**65**	**15.5**	**44.2**	**6.25**	**2.9**	**47.2**
Design Year 2032								
Belmont	80	6.2	63	3.6	9.9	6.25	0.7	10.5
Low	75	6.5	43	8.6	15.2	6.25	1.0	16.2
High	103	13.2	73	4.9	18.0	6.25	1.2	19.2
Southeast	129	7.3	86	1.2	8.5	6.25	0.6	9.1
Northwest	120	0.7	69	0.3	1.0	6.25	0.1	1.0
Cheney	128	1.1	92	0.1	1.2	6.25	0.1	1.3
Total	**96**	**35.0**	**65**	**18.7**	**53.7**	**6.25**	**3.6**	**57.3**
Design Year 2057								
Belmont	80	12.1	62	7.4	19.5	6.25	1.3	20.8
Low	75	7.5	43	10.0	17.5	6.25	1.2	18.7
High	102	16.8	70	7.2	24.0	6.25	1.6	25.6
Southeast	126	10.8	86	1.8	12.6	6.25	0.8	13.4
Northwest	120	1.2	66	0.6	1.8	6.25	0.1	1.9
Cheney	127	2.1	89	0.3	2.3	6.25	0.2	2.5
Total	**96**	**50.6**	**65**	**27.3**	**77.8**	**6.25**	**5.2**	**83.0**

Maximum day and maximum hour demands projections for each service level were evaluated based on historical peak demands and used in the *2002 Facilities Master Plan Report*. Peaking factors by class within each service level were adjusted slightly from the 2002 Facilities Master Plan so that the sum of the demands by service level would match the total system demands. Design peaking factors by class and service level are summarized in Table 3-11.

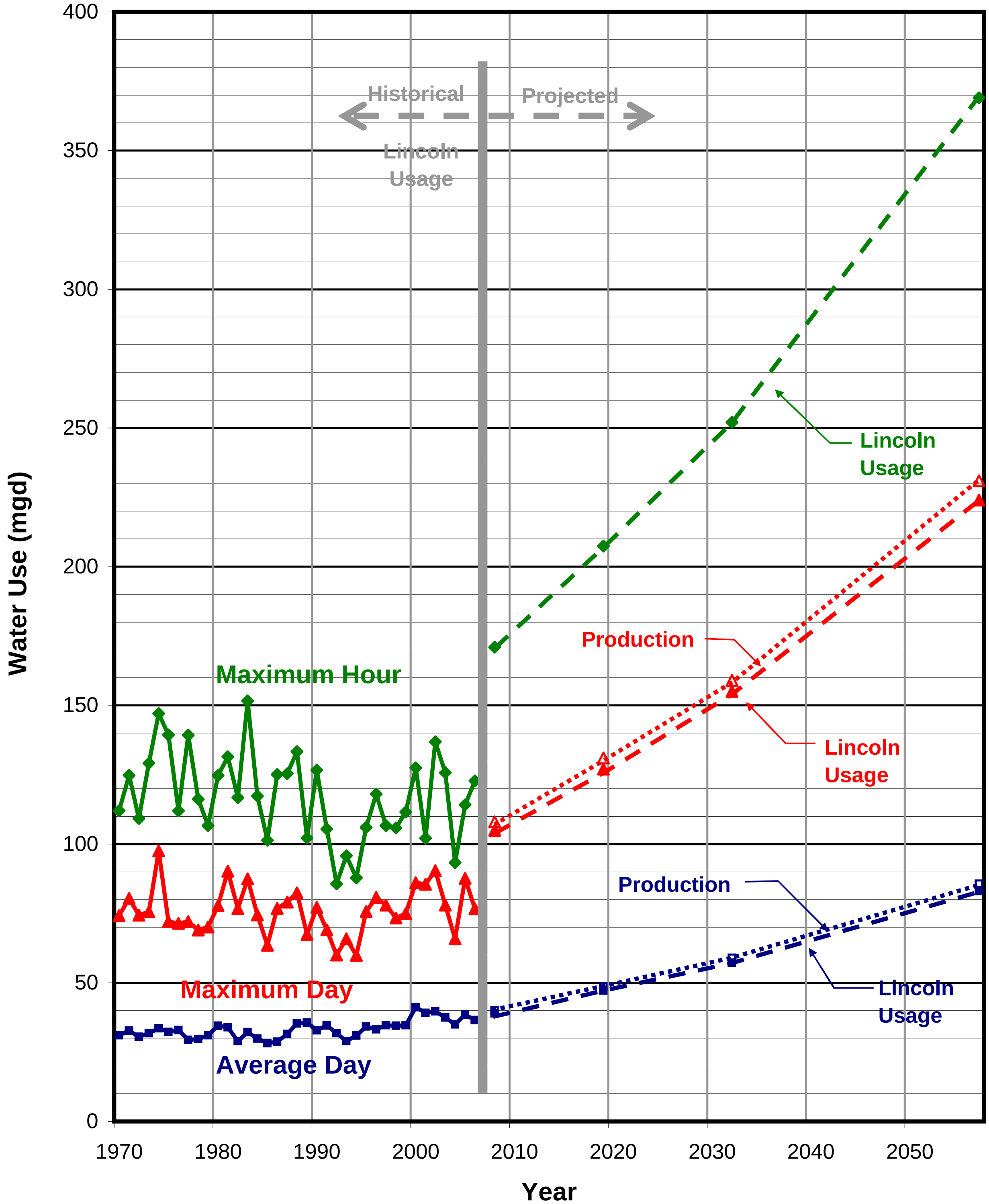
Table 3-11
Design Peaking Factors by Class by Service Level

	Residential Peaking Factors								Commercial Peaking Factors	
	Maximum Day				Maximum Hour					
Service Level	**Base**	**2019**	**2032**	**2057**	**Base**	**2019**	**2032**	**2057**	**All Years MD**	**All Years MH**
Belmont	2.6	2.6	2.6	2.6	4.4	4.4	4.4	4.4	2.5	3.1
Low	2.6	2.6	2.6	2.6	4.4	4.4	4.4	4.4	2.3	3.1
High	3.1	3.1	3.1	3.1	5.4	5.4	5.3	5.3	3	4.5
Southeast	3.3	3.2	3.1	3.1	6.2	6.1	6.1	6.1	3	4.5
Northwest	3.4	3.4	3.4	3.4	6.5	6.4	6.2	6.2	3	4.5
Cheney	3.4	3.4	3.4	3.4	6.5	6.4	6.1	6.1	3	4.5
Overall System Average	**3**	**3**	**3**	**3**	**5.2**	**5.2**	**5.2**	**5.2**	**2.40 to 2.46**	**3.49 to 3.58**
Non-revenue water peaking factor = 1.0 for all conditions and all design years.										

Design peak demands by service level are summarized in Table 3-12.

Table 3-12
Projected Water Requirements by Service Level

Service Level	Average Day (mgd)	Maximum Day (mgd)	Maximum Hour (mgd)
Base Year			
Belmont	4.7	11.6	17.5
Low	13.6	32.0	47.9
High	15.1	44.4	74.0
Southeast	4.8	14.8	27.4
Northwest	0.3	1.0	1.8
Cheney	0.4	1.1	2.1
Total	**38.8**	**104.8**	**170.8**
Design Year 2019			
Belmont	7.3	17.9	27.1
Low	15.2	35.6	53.1
High	16.7	49.1	81.8
Southeast	6.6	20.2	37.1
Northwest	0.7	2.1	3.7
Cheney	0.8	2.5	4.6
Total	**47.2**	**127.3**	**207.4**
Design Year 2032			
Belmont	10.5	25.9	39.3
Low	16.2	37.8	56.5
High	19.2	56.6	92.9
Southeast	9.1	27.0	50.7
Northwest	1.0	3.2	5.5
Cheney	1.3	4.1	7.2
Total	**57.3**	**154.7**	**252.0**
Design Year 2057			
Belmont	20.8	51.3	77.6
Low	18.7	43.8	65.4
High	25.6	75.4	123.4
Southeast	13.4	39.5	74.8
Northwest	1.9	5.9	10.1
Cheney	2.5	8.0	13.9
Total	**83.0**	**223.9**	**365.2**



Lincoln, Nebraska
2007 Facilities Master Plan Update

Historical and Projected Water Requirements

Figure 3-1

BLACK & VEATCH Corporation

4.0 Existing Water Distribution System Facilities

The LWS service area is currently divided into four major service levels - Low, High, Belmont, and Southeast. In 2001, the Cheney Booster District was created in the southeast portion of the service area to serve new development on high ground. Also in 2002, the Northwest Booster District was created near the Northwest Reservoir, to serve a new development on high ground in that area.

4.1 High Service Pumping and Transmission

The high service pumps at the water treatment plant (WTP) are located in three separate buildings. Pumps 1 through 6 are located in the North Pumping Station. Pumps 7, 8, and 9 are located in the West Pumping Station. Pumps 10, 11, and 12 are located in the South Pumping Station. Data on the WTP high service pumping units is shown in Table 4-1.

Table 4-1
WTP High Service Pumps

Pump No.	Drive Type	Rated Capacity (gpm)	Rated Capacity (mgd)	Head (ft)	Pump Motor (hp)	Pump Motor (rpm)
1	Electric – 2400 V	14,000	20.2	115	600	900
2	Electric – 2400 V	9,800	14.1	205	700	1200
3	Electric – 2400 V	9,375	13.5	205	600	1200
4	Electric – 2400 V	14,000	20.2	233	1250	900
5	Electric – 2400 V	14,000	20.2	233	1250	900
6	Electric – 2400 V	14,000	20.2	233	1250	900
7	Diesel Engine	15,000	21.6	345	1950	900
8	Electric – 4160 V	15,000	21.6	345	1750	888
9	Diesel Engine	15,000	21.6	345	1950	900
10	VFD – 480 V	14,000	20.2	130	600	710
11	VFD – 2400 V	14,500	20.9	350	1750	720
12	VFD – 2400 V	14,500	20.9	350	1750	720

Treated water from the treatment facilities is pumped to Lincoln. Under lower flow conditions, water pumped from the WTP can be delivered 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. The local wells, which pump to the "A" Street Reservoirs, are maintained as a backup water supply source and can provide additional water to the “A” Street facilities.

Additional information on the transmission system storage and pumping facilities are described later in this chapter, in the section on the Low Service Level.

4.2 Service Levels

Ground elevations within the existing service area range from about 1,130 feet (USGS datum) along Salt Creek to about 1,450 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 boundaries should have sufficient flexibility to allow minor modifications to provide adequate service, particularly at higher elevations and in developing areas. The service area is currently divided into four major service levels - Low, High, Belmont, and Southeast. In 2001, the Cheney Booster District was created in the southeast portion of the service area to serve new development on high ground. A portion of the existing Southeast Service Level was converted to the Cheney Booster District. Also in 2002, the Northwest Booster District was created near the Northwest Reservoir, to serve a new development on high ground in that area. The static hydraulic gradient for each of the four main service levels is established by the maximum water service elevation of floating storage facilities within the service area. The ground elevations served and static hydraulic gradient for each service level are shown in Table 4-2.

Table 4-2 Service Levels		
Service Level	**Ground Elevation[1] (ft)**	**Static Hydraulic Gradient Elevation (ft)**
Belmont Service Level	1150 – 1300	1,400[2]
Low Service Level	1130 – 1230	1,313[2]
High Service Level	1150 – 1320	1,420[2]
Southeast Service Level	1240 – 1390	1,500[2]
Cheney Booster District[4]	1340 – 1440	1,600[3]
Northwest Booster District[5]	1220 – 1320	1,450[3]

(1) Principal part of service level, USGS datum.
(2) Established by overflow elevation of floating storage within service level.
(3) Currently established by PRV setting at pumping station discharge.
(4) Cheney Booster District established in year 2001.
(5) Northwest Booster District established in year 2002.

4.3 Pumping Stations and System Storage

4.3.1 Low Service Level

The Low Service Level services 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 static hydraulic gradient of 1,313 feet is established by the overflow elevations of the Vine Street and Pioneers Reservoirs.

4.3.1.1 Northeast Pumping Station and Reservoir

The Northeast Reservoir is comprised of two reservoirs with a storage volume of 10.0 MG, an overflow elevation of 1,135 feet, and a sidewater depth of 18 feet.

The Northeast Pumping Station contains one transfer pump, No. 1, with a rated capacity of 31,250 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 variable speed drive allows the pumping capacity to vary from about 60-percent to 100-percent of the rated capacity at maximum speed (range of 27 mgd to 45 mgd).

The Northeast Pumping Station contains five Low Service Level distribution system pumps, Nos. 2 through 6. Pump No. 3 was installed in 1997 and Pump No. 2 was installed in

2006. Pump No. 6 is equipped with an eddy current adjustable speed drive. Both the transfer and distribution system pumps take suction from the adjacent reservoir. Data for the Northeast pumping units is given in Table 4-3.

Table 4-3
Northeast Pumping Station

Pump No.	Make	Rated Capacity (gpm)	Rated Capacity (mgd)	Head (ft)	Pump Motor (hp)	Pump Motor (rpm)
1[(1)]	Ruhrpumpen	31,250	45	60	600	705
2[(2)]	Ruhrpumpen	14,000	20.2	255	1,200	890
3[(2)]	Fairbanks	14,000	20.2	255	1,250	900
4[(2)]	Fairbanks	10,500	15.1	245	800	900
5[(2)]	Fairbanks	10,500	15.1	245	800	900
6[(2)]	Fairbanks	10,500	15.1	245	800	900

(1) Transfer pump with variable speed drive.
(2) Low Service Level distribution system pumps.

4.3.1.2 51st Street Pumping Station and Reservoirs

The 51st Street Reservoirs include a 6.0 million gallon, 5.0 million gallon, and 1.0 million gallon ground storage reservoirs supplied from the water treatment plant. The 5.0 and 1.0 million gallon reservoirs have overflow elevations of 1,148 feet and sidewater depths of 14.2 feet. The 6.0 million gallon reservoir has an overflow elevation of 1,148 feet and a sidewater depth of 15.33 feet.

The 51st Street Pumping Station contains three transfer pumps, Nos. 1 through 3. 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. The transfer pumps discharge to a low pressure transmission/transfer main which extends to the "A" Street Reservoirs. The 51st Street Pumping Station contains four Low Service Level distribution system pumps, Nos. 4 through 7. 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. Both the transfer and distribution system pumps take suction from the 51st Street Reservoirs. Data on the 51st Street pumping units is given in Table 4-4.

Table 4-4 51st Street Pumping Station						
Pump No.	**Make**	**Rated Capacity**		**Head**	**Pump Motor**	
		(gpm)	**(mgd)**	**(ft)**	**(hp)**	**(rpm)**
1(1)	Ingersoll-Dresser	10,500	15.1	185	750	900
2(1)	Ingersoll-Dresser	10,500	15.1	185	750	900
3(1)	Ingersoll-Dresser	10,500	15.1	185	750	900
4(2)	Ingersoll-Dresser	7,000	10.1	230	500	900
5(2)	Ingersoll-Dresser	7,000	10.1	230	500	900
6(2)	Ingersoll-Dresser	7,000	10.1	230	500	900
7(2)	Ingersoll-Dresser	7,000	10.1	230	500	900

(1) Transfer pumps – new in 2001.
(2) Low Service Level distribution system pumps – new pumps and motors in 2001.

4.3.1.3 "A" Street Pumping Station and Reservoirs

The "A" Street Reservoirs are comprised of six ground storage reservoirs have a total capacity of 32.0 million gallons which are supplied from several locations. The reservoirs can also be supplied from the local wellfield. The six reservoirs have different overflow elevations but are interconnected and float together establishing a common hydraulic gradient. Data on the "A" Street Reservoirs is shown in Table 4-5.

Table 4-5 "A" Street Reservoirs			
Reservoir No.	**Capacity MG**	**Ceiling or Overflow Elevation (ft)**	**Floor Elevation (ft)**
4	2.0	1183.2	1166.7
5	4.0	1186.4	1170.4
6	6.0	1190.8	1174.8
7	4.0	1192.5	1170.5
8	8.0	1190.8	1171.5
9	8.0	1190.1	1175.0

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, Nos. L1 and L2, each with a rated capacity of 6,300 gpm (9.0 mgd) at 155 feet. Three "satellite" pumps are located at the "A" Street facilities in three separate buildings. Satellite 8 discharges to the Low Service Level. Satellites 9 and 10 discharge to the High Service Level. All Low and High Service Level pumps take suction from the adjacent reservoirs. Data on the "A" Street pumping units is shown in Table 4-6.

Table 4-6
"A" Street Pumping Station

Pump No.	Make	Rated Capacity (gpm)	Rated Capacity (mgd)	Head (ft)	Pump Motor (hp)	Pump Motor (rpm)
Low Service Level						
L1[1]	Patterson	6,300	9.1	155	350	1,200
L2[1]	Patterson	6,300	9.1	155	350	1,200
Sat. 8[1]	Flowserve	7,200	10.4	155	450	1,200
High Service Level						
H1[2]	Patterson	6,300	9.1	265	600	1,800
H2[2]	Patterson	6,300	9.1	265	600	1,800
Sat. 9[2][3]	Layne	6,300	9.1	250	500	1,200
Sat. 10[2][3]	Fairbanks- Pomona	6,300	9.1	250	500	1,200

(1) Low Service Level.
(2) High Service Level.
(3) Pump motors are currently planned for rehabilitation.

4.3.1.4 Vine Street Reservoir

The Vine Street Reservoir was expanded from 10.0 MG to 20.0 MG in 2001. It floats on the Low Service Level, has an overflow elevation of 1,313 feet, and a sidewater depth of 30 feet. The reservoir also provides suction storage for the adjacent Vine Street Pumping Station, which supplies the High and Southeast Service Levels.

4.3.1.5 Pioneers Reservoir

The Pioneers Reservoir is a four million gallon reservoir which floats on the Low Service Level, has an overflow elevation of 1,313 feet, and a sidewater depth of 54 feet.

4.3.2 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. The High Service Level static hydraulic gradient of 1,420 feet is established by the overflow elevations of the Southeast and Pine Lake Reservoirs.

4.3.2.1 "A" Street Pumping Station

The "A" Street Pumping Station contains two High Service Level pumps, Nos. H1 and H2, each with a rated capacity of 6,300 gpm (9.0 mgd) at 265 feet. The "A" Street facilities also contains two satellite pumping stations, Nos. 9 and 10, each with a rated capacity of 6,300 gpm (9.0 mgd) at 250 feet that discharge to the High Service Level. Data on the "A" Street pumping units was shown earlier in this chapter in Table 4-6.

4.3.2.2 Vine Street Pumping Stations

The Vine Street Pumping Stations are located at the Vine Street Reservoir site and take suction from the Vine Street Reservoir.

The High Service Level station contains four pumps. Pump No. 1 has a rated capacity of 10,500 gpm (15.0 mgd) at 115 feet and is equipped with a variable speed drive. Pump Nos. 2 through 4 have a rated capacity of 14,000 gpm (20.2 mgd) at 115 feet. There is space available for a fifth pump.

The Southeast Service Level 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. One variable speed drive is located in the station and can be used to operate either of the two pumps by the use of a transfer switch. There is space available for a third pump. The facility is designed to accommodate 20 mgd pumps in each of the three pump slots.

Data on the Vine Street pumping units is shown in Table 4-7.

Table 4-7
Vine Street Pumping Stations

Pump No.	Make	Rated Capacity (gpm)	Rated Capacity (mgd)	Head (ft)	Pump Motor (hp)	Pump Motor (rpm)
High Service Level						
H1[(1)]	Worthington	10,500	15.1	115	400	870[(3)]
H2[(1)]	Worthington	14,000	20.2	115	500	1,175
H3[(1)]	Worthington	14,000	20.2	115	500	1,175
H4[(1)]	Worthington	14,000	20.2	115	500	1,175
Southeast Service Level						
SE1[(2)]	Ingersoll	7,000	10.1	210	450	895[(4)]
SE2[(2)]	Ingersoll	7,000	10.1	210	450	895[(4)]

(1) High Service Level.
(2) Southeast Service Level.
(3) Variable speed drive.
(4) Common variable speed drive can be used for either of the two Southeast pumps.

4.3.2.3 Pine Lake Reservoir and Pumping Station

The Pine Lake Reservoir is a 4.0 million gallon reservoir which floats on the High Service Level, has an overflow elevation of 1,420 feet, and a sidewater depth of 62 feet.

In 1998 a re-pumping station was added at the reservoir site. The station contains three pumps each rated 3,125 gpm (4.5 mgd) at 50 feet. The pumping station is intended to be used to increase pressures in the southern portion of the High Service Level under high flow conditions. Records indicate that the station is not used (other than exercising of the pumps). Data on the Pine Lake pumping units is shown in Table 4-8.

Table 4-8
Pine Lake Pumping Station

Pump No.	Make	Rated Capacity		Head	Pump Motor	
		(gpm)	(mgd)	(ft)	(hp)	(rpm)
1	General Signal	3,125	4.5	50	50	1,170
2	General Signal	3,125	4.5	50	50	1,170
3	General Signal	3,125	4.5	50	50	1,170

4.3.2.4 Southeast Reservoir

The Southeast Reservoir is a 5.0 million gallon reservoir which floats on the High Service Level, has an overflow elevation of 1,420 feet, and a sidewater depth of 60 feet. The reservoir also provides suction storage for the adjacent Southeast Pumping Station, which supplies the Southeast Service Level.

4.3.3 Belmont Service Level

The Belmont Service Level serves the northwest part of the City, including 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 Reservoirs.

4.3.3.1 Belmont Pumping Station

The Belmont Pumping Station is located southwest of the intersection of 14th and Superior Streets. The Belmont Pumping Station takes suction from Low Service Level mains and contains four pumps.

The impeller in Pump No. 1 was replaced in 1999. A new impeller was installed in Pump No. 2 in 1990. Pump No. 3 was replaced in 2001 and Pump No. 4 was installed in 1990. A shared adjustable frequency drive for Pump Nos. 2 and 4 was removed in 2001. Data on the Belmont pumping units is shown in Table 4-9.

Table 4-9 Belmont Pumping Station						
		Rated Capacity		Head	Pump Motor	
Pump No.	Make	(gpm)	(mgd)	(ft)	(hp)	(rpm)
1	Allis-Chalmers	4,200	6.1	135	200	1,170
2	Allis-Chalmers	4,200	6.1	135	200	1,170
3	Ingersoll-Dresser	6,300	9.1	135	300	1,185
4	Allis-Chalmers	6,300	9.1	135	300	1,185

4.3.3.2 Merrill Pumping Station

The Merrill Pumping Station takes suction from the 51st Street Pumping Station. As part of the 2001 pumping station modifications project, a shared adjustable frequency drive was removed, and constant speed motors were installed on both units, allowing both to be operated at the same time. Data on the Merrill pumping units is shown in Table 4-10.

Table 4-10 Merrill Pumping Station						
		Rated Capacity		Head	Pump Motor	
Pump No.	Make	(gpm)	(mgd)	(ft)	(hp)	(rpm)
1	Allis-Chalmers	2,600	3.7	215	200	1,760
2	Allis-Chalmers	2,600	3.7	215	200	1,760

4.3.3.3 Pioneers Pumping Station

The Pioneers Pumping Station contains three pumps that boost from the Low Service Level to the Belmont Service Level. There is space for addition of a fourth pump in the station. Data on the Pioneers Pumping Station is shown in Table 4-11.

Table 4-11 Pioneers Pumping Station						
		Rated Capacity		Head	Pump Motor	
Pump No.	Make	(gpm)	(mgd)	(ft)	(hp)	(rpm)
1	Fairbanks-Morse	1,400	2.0	105	60	1,195
2	Fairbanks-Morse	2,100	3.0	105	75	1,190
3	Fairbanks-Morse	3,500	5.0	105	125	1,185

4.3.3.4 Air Park Reservoir

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

4.3.3.5 Northwest Reservoir

The Northwest Reservoir is a 4.5 million gallon reservoir which floats on the Belmont Service Level, has an overflow elevation of 1,400 feet, and a sidewater depth of 75 feet.

4.3.4 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 location. The Southeast static hydraulic gradient of 1,500 feet is currently established by the overflow elevation of the Yankee Hill Reservoir. Upon completion of the Yankee Hill Reservoir in 2003, the Pine Lake Reservoir which previously served the Southwest Service Level was demolished.

4.3.4.1 Southeast Pumping Station

The Southeast Pumping Station has four pumping units. The impeller in Pump No. 1 was replaced in 1999. A new impeller was installed in Pump No. 2 in 1999. Pump No. 3 was replaced in 2001 and Pump No. 4 was installed in 1988. A shared adjustable frequency drive for Pump Nos. 2 and 4 was removed in 2001. Data on the Southeast pumping units is shown in Table 4-12.

Table 4-12
Southeast Pumping Station

Pump No.	Make	Rated Capacity (gpm)	Rated Capacity (mgd)	Head (ft)	Pump Motor (hp)	Pump Motor (rpm)
1	Allis-Chalmers	4,200	6.1	155	200	1,180
2	Allis-Chalmers	4,200	6.1	155	200	1,170
3	Ingersoll-Dresser	6,300	9.1	155	350	1,185
4	Allis-Chalmers	6,300	9.1	155	350	1,185

4.3.4.2 Yankee Hill Reservoir

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

4.3.5 Cheney Booster District

The Cheney Booster District 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 Booster District. The Cheney Booster Pumping Station (BPS) was installed in 2001. The Cheney Booster District initially operated as a closed system with no floating storage. The static hydraulic gradient in the booster district is established by the pressure reducing valve (PRV) setting on the pumping station discharge. A pressure set-point of 91 psi equates to an hydraulic gradient of 1,600 feet. When the Cheney Reservoir comes online the static hydraulic gradient in the Cheney Booster District will be reduced to 1,580 feet.

4.3.5.1 Cheney Booster Pumping Station

The Cheney Booster Pumping Station is a pre-packaged below-grade pumping station containing five pumps. Data on the Cheney pumping units is shown in Table 4-13.

Table 4-13
Cheney Booster Pumping Station

Pump No.	Make	Rated Capacity (gpm)	Rated Capacity (mgd)	Head (ft)[(1)]	Pump Motor (hp)	Pump Motor (rpm)
1	Paco	130[(2)]	0.2	175	10	3,500
2	Paco	650	0.9	175	40	3,500
3	Paco	1,400	2.0	175	100	1,750
4	Paco	2,150	3.1	175	125	1,750
5	Paco	2,150	3.1	175	125	1,750

(1) Although pumps are rated at 175 feet of head, the discharge PRV throttles about 75 feet of head at 91 psi to maintain a hydraulic gradient of about 1600 feet.
(2) "Jockey" pump used only for very low flow conditions.

4.3.5.2 Cheney Reservoir

At the writing of this report the Cheney Reservoir had just been commissioned for service. The 2.0 million gallon reservoir floats on the Cheney Service Level and has an overflow elevation of 1,580 feet and a sidewater depth of 40 feet.

4.3.6 Northwest Booster District

The Northwest Booster District was placed into service in 2002 to serve new development on high ground in the northern portion of the city. The Northwest Booster Pumping Station (BPS) was installed in 2002 at the Northwest Reservoir site. The Northwest Booster District is operated as a closed system with no floating storage. The static hydraulic gradient in the booster district is established by the pressure reducing valve (PRV) setting on the pumping station discharge. LWS reports that the valve set-point is set at 61 psi equates which equates to an hydraulic gradient of about 1,460 feet.

4.3.6.1 Northwest Booster Pumping Station

The Northwest Pumping Station is a pre-packaged above-grade pumping station containing five pumps. Data on the Northwest pumping units is shown in Table 4-14.

Table 4-14
Northwest Booster Pumping Station

Pump No.	Make	Rated Capacity (gpm)	Rated Capacity (mgd)	Head (ft)[1]	Pump Motor (hp)	Pump Motor (rpm)
1	Paco	150[2]	0.2	100	7.5	3,600
2	Paco	650	0.9	100	25	1,800
3	Paco	1,400	2.0	100	50	1,800
4	Paco	2,200	3.2	100	75	1,800
5	Paco	2,200	3.2	100	75	1,800

(1) Although pumps are rated at 100 feet of head, the discharge PRV throttles about 50 feet of head at 54 psi to maintain a hydraulic gradient of about 1450 feet.
(2) "Jockey" pump used only for very low flow conditions.

4.3.7 Pumping Capacity Summary

A summary of total and firm capacities for existing distribution system pumping stations is summarized in Table 4-15.

Table 4-15
Distribution System Pumping Capacity Summary

Service Level	Pumping Station	Number of Pumps	Installed Capacity (mgd)	Firm Capacity (mgd)
Low	51st Street[1]	4	40.4	30.3
	Northeast[1]	5	85.7	65.5
	"A" Street	3	28.6	18.2
	Total		**134.5**	**93.8**
High	"A" Street	4	36.4	27.3
	Vine Street	4	75.7	55.5
	Pine Lake	3	13.5	9.0
	Total		**125.6**	**91.8**
Belmont	Belmont	4	30.4	21.3
	Merrill	2	7.4	3.7
	Pioneers	3	10.0	5.0
	Total		**47.8**	**30.0**
Southeast	Vine Street	2	20.2	10.1
	Southeast	4	30.4	21.3
	Total		**50.6**	**31.4**
Cheney	Cheney	4[2]	9.1	6.0
Northwest	Northwest	4[2]	9.3	6.1

(1) Transfer pumps not included.
(2) Does not include capacity of small "jockey" pump.

4.3.8 Storage Capacity Summary

A summary of floating storage capacities by service level is given in Table 4-16. It is noted that a number of reservoirs provide both floating storage and suction storage to different service levels.

Table 4-16
Distribution System Floating Storage Capacity Summary

Service Level	Reservoir	Capacity MG
Low	Vine Street	20.0
	Pioneers	4.0
	Total	**24.0**
High	Southeast	5.0
	High Service	4.0
	Total	**9.0**
Belmont	Air Park	3.0
	Northwest	4.5
	Total	**7.5**
Southeast	Yankee Hill	10.0
	Total	**10.0**
Cheney	Cheney	2.0
	Total	**2.0**
	Grand Total	**53.5**

Additional ground storage is provided on the transmission system from the water treatment plant for repumping to the distribution system as summarized in Table 4-17.

Table 4-17
Transmission Ground Storage Facilities

Reservoir	Capacity (MG)
Northeast	10.0
51st Street	12.0
"A" Street	32.0
Total	**54.0**

The total storage volume, including the transmission ground storage facilities and the distribution system floating storage, is 107.5 MG.

5.0 Distribution System Analyses

5.1 General

Hydraulic analyses were conducted to evaluate the Lincoln distribution system, and to establish an improvement program to reinforce the existing system and allow expansion to meet projected water demands through the year 2057. Alternative improvements were investigated to identify those most effective in meeting projected water demands. Criteria used to develop the improvement program include increasing system reliability, simplifying system operations, more effectively utilizing system storage to meet peak demands, and maintaining minimum pressures under maximum hour demand conditions. This section discusses development of the hydraulic computer model and results of the analyses performed.

Computer hydraulic analysis is a method of predicting the hydraulic gradient pattern, pressures, and flows across the water distribution network under a given set of conditions. The hydraulic gradient pattern 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 pressure or head, at any point in the network is the difference between the hydraulic gradient and the ground elevation.

5.2 Computer Model

As part of this update the Lincoln water distribution system was evaluated using the network analysis program, H2OMAP, which operates in ESRI MapObject environment. The modeling software can easily display ArcGIS shapefiles that can be exported from the file Geodatabase structure used by several departments in the City including the Planning Department.

The physical characteristics of the water distribution system in the computer model include ground topography, reservoir elevations, pump characteristics, and pipe diameter, length, and interior roughness. Historical and projected water demands are also assigned to the computer model. The model contains all gridded, or looped, mains of 4-inch diameter and greater. Dead-end mains of 12-inch diameter and greater were typically included in the model, unless they were of short length of about 100 feet or less.

The computer model of the Lincoln water distribution system was updated from the 2002 water model using current GIS data provided by LWS. The original 2002 model was created from an electronic line drawing (Microstation format) of the year 2000 distribution system. The drawing was converted to an ESRI ArcInfo coverage and processed into a coherent map of node and line elements. The model includes all service levels and the transmission system from the water treatment plants to the Lincoln 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.

5.2.1 Pipe Friction Coefficient (C-value)

The pipe friction coefficient, "C" value in the Hazen-Williams empirical equation for pipe flow, is an index of pipe hydraulic capacity. 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. The typical "C" value for a new cement-lined ductile iron pipe is about 130, and decreases as pipes age. Prior to the 1960's mains were generally not lined with cement mortar, resulting in tuberculation and lower "C" values.

The mains in the Lincoln water distribution system are mostly lined cast or ductile iron and are generally in good condition. The "C" values assigned in the 2002 computer model were maintained in the updated model. “C” values ranged from 140 for newer large diameter transmission mains, to 70 for older and smaller mains in the distribution system. All future mains were modeled with a “C” value of 130.

5.2.2 Demand Allocation

An updated demand allocation was conducted to replace the 2002 model demand allocation. The H2OMAP computer model enables allocation of demands to junctions based on up to ten classes or fields. Base year demands were allocated to the computer model using existing metered sales data and applying the population and water requirement design values discussed in details in Chapters 2 and 3. The base year allocation utilized the first 4 user class fields (designated as residential, non-residential, large user, and non-revenue). Future year demand allocation was based on TAZ population data and the water requirement design values discussed in Chapter 3. The demands can be factored based on geographical variations in water use, allowing a broad range of demand conditions to be simulated.

5.2.2.1 Base Year Demand Allocation

Base year average day demands of 38.8 mgd (calculated by applying design per-capita use rates multiplied by the year 2006 population) were allocated to the model of the existing distribution system. LWS provided fiscal year 2006 geocoded metered sales information for every account in the LWS Distribution System. The information included account number, account address, annual sales in 100-cubic-feet (ccf), and user classification code. The account number identified the meter cycle for each account. The data consisted of a total of 79,097 records. 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 2006. Non-revenue water was allocated to each node in the model based on the percent of total demand at each node.

5.2.2.2 Year 2019, 2032, and 2057 Demand Allocation

Future residential demands were allocated to the model based on population 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 demands for every TAZ and TAZ segment. Design ratios for the breakdown of residential to non-residential demands by zone as described in Chapter 3 were used to determine the total demand for each TAZ segment. The beginning per-capita water use rates used to calculate future residential demands are shown in Table 5-1. The per-capita rates were reduced slightly for each design year in the calculation of demands for the High SL, Southeast SL, and the Cheney Service Level but the maximum decrease in per-capita use rates was no more than 7 gpcd.

Table 5-1 Service Level Per-capita Residential Use Rates	
Service Level	**Per-capita Residential Use Rate (gpcd)**
Belmont	80
Low	75
High	105
Southeast	133
Northwest	120
Cheney	132

The thiessen polygon method was used to allocate the demand 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 demand contained by the 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 demands were allocated to the model based on a design non-revenue demand of 6.25 percent of the total water use. Non-revenue water demands were allocated to nodes with residential or non-residential demands at a value equal to 6.25 percent of the total demand.

5.2.2.3 Model Calibration

Recorded SCADA data was used to determine demand diurnals for each service level for the day of calibration. The day that was selected for use in the model calibration was July 19, 2006 the day of maximum daily demand for that year. The simulation was performed and slight changes were iteratively made until the results closely resembled the SCADA data provided. In summary, the LWS distribution model was consistently among the best in class when compared to the calibration statistical accuracies of seven other case studies reported in *Quantitative Results of EPS Model Calibrations with a Comparison to Industry Guidelines* from the November, 2007 issue of the Journal AWWA.

5.3 Base Year Steady State Analyses

A series of analyses was conducted under base year design demands and existing deficiencies were evaluated in conjunction with immediate improvements relating to the LWS 6-year capital improvements program (CIP). The immediate improvements necessary to correct existing deficiencies are categorized as Phase I recommended improvements.

Maximum day, maximum hour, and replenishment steady state hydraulic simulations were performed in order to verify the distribution response under design conditions applied to the existing system.

5.3.1 Base Year Maximum Day Analysis

The purpose of a maximum day steady state analysis is to ensure that for a 24-hour period of maximum day demand the total volume of water contained in distribution storage can be maintained while pumping stations are constrained to operate at or below their firm capacity. This criterion is considered to be achieved in a steady state simulation when the total inflow/outflow results for each storage facility approaches zero. The base year maximum day analysis indicates that no pumping station improvements are necessary to meet design maximum day demand conditions while maintaining the total storage volume over the course of a day.

5.3.2 Base Year Maximum Hour Analysis

One purpose of a maximum hour steady state analysis is to review the theoretical system pressures under design maximum hour demand conditions. Pressures lower than the minimum goals indicate that system head losses are larger than desired and improvements are necessary in the distribution piping to reduce these head losses and increase pressures.

Another objective of the maximum hour analysis is to verify that the contribution rate for storage facilities remains below the maximum acceptable contribution rate. As outlined in the *2002 Facilities Master Plan* report, it is accepted industry practice to utilize the top one-third of floating storage capacity to satisfy peak demands for ground storage and up to the top one-half of floating storage for elevated reservoirs. The remainder of the storage volume is reserved for fire fighting and emergencies. For the hydraulic analyses performed, it was assumed that the peak demand experienced during the maximum hour condition had a duration of 4 hours. Based on these assumptions the maximum contribution rate during the maximum hour scenario is tabulated in Table 5-2 along with the utilization rate resulting from the maximum hour simulation.

Table 5-2
Base Year Maximum Hour - Storage Contribution and Utilization

Reservoir[1]	Capacity (MG)	Maximum Acceptable Contribution[2] (mgd)	Contribution (mgd)	Utilization[3] (%)
Vine Street	20.0	40.0	14.6	36
Pioneers	4.0	8.0	3.5	43
Pine Lake	4.0	8.0	8.0	100
Southeast	5.0	10.0	5.3	53
Air Park	3.0	6.0	2.0	34
Northwest	4.5	9.0	4.1	45
Yankee Hill	10.0	20.0	15.3	76
Cheney Reservoir	2.0	6.0[4]	0.7	11

(1) Storage Reservoirs within the transmission system (51st Street, Northeast, and "A" St) are not included.
(2) Maximum acceptable contribution based on the top one-third of reservoir volume depleted over a 4-hour duration.
(3) Utilization represents maximum hour contribution as a percent of maximum acceptable contribution.
(4) Maximum acceptable contribution for Cheney Reservoir based on one-half rather than one-third of reservoir volume depleted over a 4-hour duration because it is elevated.

5.3.3 Base Year Replenishment Analysis

Maximum distribution system pressures typically occur during the period of storage replenishment because demands are low but pumps continue operating in order to replenish system storage. Recorded SCADA data from the week of July 17-23, 2006 was reviewed to determine peaking factors during the daily replenishment times in peak demand periods. Average peaking factors during the time period of 11 pm to 5 am were reviewed and a replenishment peaking factor was calculated and applied to the model to obtain typical replenishment demands.

Another important consideration during review of the replenishment analysis results is to determine whether the refill rate (inflow) allows for full replenishment of storage given the utilization rate (outflow) resulting from the maximum hour analysis. The assumption that storage utilization occurs for a period of 4 hours and that storage refill occurs for a period of 6 hours yields the solution that the required rate at which the storage needs be replenished is greater or equal to two-thirds the outflow rate obtained from the maximum hour simulation. Table 5-3 illustrates the refill rates and the refill capabilities.

Table 5-3 Base Year Replenishment Scenario – Storage Refill Capability					
Reservoir(1)	Capacity (MG)	Maximum Hour Simulation Draft Rate (mgd)(2)	Required Refill Rate (mgd)(3)	Replenishment Simulation Calculated Refill Rate (mgd)	% Refill Capability(4)
Vine Street	20.0	14.6	9.7	13.3	137
Pioneers	4.0	3.5	2.3	3.7	161
Pine Lake	4.0	8.0	5.3	11.0	208
Southeast	5.0	5.3	3.5	5.9	169
Air Park	3.0	2.0	1.3	1.3	100
Northwest	4.5	4.1	2.7	2.7	100
Yankee Hill	10.0	15.3	10.2	10.3	101
Cheney Reservoir	2.0	0.7	0.5	0.6	120

(1) Storage Reservoirs within the transmission system (51st St, Northeast, and “A” St) are not included.
(2) Maximum Hour Simulation Draft Rate obtained from the results of the maximum hour analyses.
(3) Required refill rate is equal to two-thirds the maximum hour simulation draft rate.
(4) Percent Refill Capability represents the simulated refill rate as a percent of the calculated required refill rate.

The “Refill Capability” is only a relative indicator of the simplicity by which a reservoir can be filled but this value may not be the ultimate indicator depending on whether pumping stations supplying the zone are pumping at maximum capacity. In these analyses the pumping was kept at a maximum day rate and not necessarily the pumping station firm capacity. For example, the value of 100% for Air Park might indicate that the reservoir is just capable of refilling in the duration of refill, but as the Belmont Pumping Station is only pumping about half of its maximum firm capacity the theoretical capability could be greater. A concern would only arise if any of the values were less than 100 percent and the pumping stations supplying the service level were at maximum firm capacity. A review of the refill capabilities and pumping station capacities show that there are no concerns in the ability to replenish storage volumes.

5.4 Year 2057 Long-Range Analyses

A series of analyses were conducted under year 2057 design demands, and included main improvements required to serve the year 2057 service limits. Before any hydraulic analyses were performed for this design year, a conceptual approach of balancing demands within the system by the transfer of water through the transmission system and existing pumping stations was performed. Where the mass balance was not capable of satisfying by existing facilities and transmission piping, future pumping stations and transmission lines were including in the conceptual plan.

During the conceptualization of the supply balance for the long-range plan, areas of high ground located near existing pressure zone boundaries were reviewed to determine the adequacy of serving these locations within the proximal service level. When necessary, future booster districts were delineated along with the future boundaries of existing service levels. The controlling notion behind the task of delineating future service level boundaries is to meet minimum pressure guidelines of 40 psi during peak demands as discussed previously in this chapter.

Once the conceptual plan had been developed, hydraulic analyses were performed to determine the adequacy of the concept. Slight alterations were made to the conceptual plan, generally to the pipe diameters, until pressure constraints and transmission capabilities were adequately satisfied. The phasing of these improvements was determined based on the results of the base year, year 2019, and year 2032 hydraulic analyses.

5.5 Year 2019 Analyses

A series of analyses were conducted under year 2019 design demands, and included main improvements required to serve the year 2019 service limits. The year 2019 analyses included completion of transmission mains currently in construction from the water treatment plant, as well as a new transmission main between Northeast Reservoirs and Vine Street Reservoir. Additionally, distribution system gridding to support development (down to 12-inch diameter mains on a quarter-mile grid, except when the piping is a portion of a future project that requires larger diameters), and the following pumping station improvements were included:

- A third pumping unit at the Vine Street Southeast Pumping Station with a unit similar to the two existing units and rated at 20 mgd. This would increase the firm pumping capacity to the Southeast Service Level from the Vine Street Southeast Pumping Station to 20 mgd.
- A new satellite pumping station to deliver water from the “A” Street Reservoirs to the Low Service Level. The pump would be rated at 10 mgd. This increases the firm pumping capacity to the Low Service Level from the “A” Street location to 28 mgd.
- Installation of Pump No. 13 at the WTP with a pump similar to Pump No. 11 and Pump No. 12. This would increase the theoretical firm capacity at the WTP for the high head pumps (Pumps number 7, 8, 9, 11, 12, and 13) to about 107 mgd.

- A small booster pumping station to provide service to the development area located north of I-80 between 40th and 56th Streets.

Table 5-4 summarizes the storage utilization versus the maximum contribution rate for the year 2019 analyses.

Table 5-4
Year 2019 Maximum Hour - Storage Contribution and Utilization

Reservoir[1]	Capacity (MG)	Maximum Acceptable Contribution[2] (mgd)	Contribution (mgd)	Utilization[3] (%)
Vine Street	20.0	40.0	16.7	42
Pioneers	4.0	8.0	3.4	42
Pine Lake	4.0	8.0	7.6	95
Southeast	5.0	10.0	10.4	104
Air Park	3.0	6.0	5.3	88
Northwest	4.5	9.0	3.7	41
Yankee Hill	10.0	20.0	18.7	94
Cheney Reservoir	2.0	6.0[4]	2.3	39

(1) Storage Reservoirs within the transmission system (51st St, Northeast, and “A” St) are not included

(2) Maximum acceptable contribution based on the top one-third of reservoir volume depleted over a 4-hour duration

(3) Utilization represents maximum hour contribution as a percent of maximum acceptable contribution

(4) Maximum acceptable contribution for Cheney Reservoir based on one-half rather than one-third of reservoir volume depleted over a 4-hour duration because it is elevated.

5.6 Year 2032 Analyses

A series of analyses was conducted under year 2032 design demands, and included main improvements required to serve the year 2032 service limits. The year 2032 hydraulic analyses include additional distribution mains to serve development (generally 16-inch diameter mains on a 1-mile grid, except when the piping is a portion of a future project that requires larger diameters), pumping station improvements, and two new storage facilities. The 2032 facility improvements added to the model for the hydraulic analyses are summarized below:

- Replacement of one 10 mgd pump at the Vine Street Southeast Pumping Station with a unit rated at 20 mgd. This would increase the firm pumping capacity to the

Southeast Service Level from the Vine Street Southeast Pumping Station to 30 mgd.

- Addition of a fourth pumping unit at the Pioneers Pumping Station rated at 5 mgd. This would increase the firm pumping capacity of the Pioneers Pumping Station to 10 mgd.
- Replacement of the WTP Pump 10 with a pump similar to Pump 11 and Pump 12 and addition of Pump No. 14 pump similar to Pump No. 11 and Pump No. 12 for a total high service firm capacity of about 148 mgd. The actual operating capacity of 7 high service pumps would be greater than 148 mgd due to a reduction in the required total discharge head.
- Addition of a new pumping station to replace the existing Cheney Pumping Station to provide service to the Cheney Booster District.
- Addition of a 4.0 MG storage facility for the South Portion of the High Service Level. This facility would help to meet maximum hour demands in the High Service Level, increase localized pressures, and provide reliability in the case of emergency.
- Addition of a 5.0 MG storage facility for the Southwest portion of the Belmont Service Level. This facility would help to meet maximum hour demands in the Belmont Service Level, increase localized pressures, and provide reliability in the case of emergency.

Table 5-5 summarizes the storage utilization versus the maximum contribution rate for the year 2032 analyses.

Table 5-5
Year 2032 Maximum Hour - Storage Contribution and Utilization

Reservoir[1]	Capacity (MG)	Maximum Acceptable Contribution[2] (mgd)	Contribution (mgd)	Utilization[3] (%)
Vine Street	20.0	40.0	16.4	41
Pioneers	4.0	8.0	4.2	53
Pine Lake	4.0	8.0	8.0	100
Southeast	5.0	10.0	9.8	98
Air Park	3.0	6.0	5.4	89
Northwest	4.5	9.0	3.6	41
Yankee Hill	10.0	20.0	19.3	97
Cheney Reservoir	2.0	6.0[3]	3.0	50
Future Saltillo Road Reservoir	4.0	8.0	8.0	100
Future Southwest Storage Facility	5.0	10.0	4.7	47

(1) Storage Reservoirs within the transmission system (51st St, Northeast, and "A" St) are not included

(2) Maximum acceptable contribution based on the top one-third of reservoir volume depleted over a 4-hour duration

(3) Utilization represents maximum hour contribution as a percent of maximum acceptable contribution

(4) Maximum acceptable contribution for Cheney Reservoir based on one-half rather than one-third of reservoir volume depleted over a 4-hour duration because it is elevated.

5.7 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.

Detailed fire flow evaluations were performed during this update and the results were incorporated into the development of improvements program. A memorandum detailing the hydraulic modeling and GIS processes performed for this task is found in Appendix A – Fire Flow Deficiency Analyses. Improvements necessary to remediate the potential deficiencies discovered during these analyses were incorporated into Chapter 8.0 – Recommended Improvements of this report.

5.8 Natural Disaster Preparedness Analyses

A series of analyses was performed in conjunction with the master plan update to determine the performance of the distribution system during various proposed emergency conditions. These analyses utilized the base year hydraulic scenario with modifications to the magnitudes of demand using values of 30 mgd and 20 mgd, about 1/3rd of the demand of the base year maximum day demands. The spatial distribution remained identical to the existing distribution of demands. The findings of these analyses were reviewed and incorporated into the planning efforts. The observations of greatest significance are summarized below:

- There is a remote modulation operation valve between the Southeast Service Level transmission main, at the Southeast reservoir that may be necessary to utilize in an emergency situation.
- The High Service Level presents the largest vulnerability when considering the amount of storage versus the service level demand. It may be necessary in an emergency to greatly utilize the pressure reducing valves between the Southeast Service Level and the High Service Level. While transfer through these valves were considered in other hydraulic analyses performed, it would not normally be necessary to transfer flow during periods of lower demand.

5.9 Hydraulic Analyses Observations

5.9.1 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. 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. Improvements are not recommended in this report to correct all low pressure areas identified in the hydraulic analyses. 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 possibly install pressure monitors to determine precise conditions in the low pressure areas. 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 simulation indicate that pressures less than 30 psi occur in the Southeast Service Level in the area between Pine Lake and Yankee Hill. Not only are these pressures lower than the minimum goal set as criteria in this analysis but they also make it difficult to transfer needed flow from the Southeast Service Level to the High Service Level at the PRV at the Pine Lake Reservoir site during peak demand periods. Transfer would be required at this location during peak demand periods in order to avoid over drafting Pine Lake Reservoir beyond the maximum allowable contribution rate. The improvement main on Yankee Hill connecting the distribution system would remediate these low pressures and the corresponding difficulty in transferring water through the PRV would be eliminated.

There are several areas that result in high pressures (greater than 110 psi) from the replenishment simulation. High pressures are found in the following areas:

- Between South Street and Van Dorn from 8th Street to 15th Street.
- Between “A” Street and Van Dorn from West 32nd Street to the Homestead Expressway.
- The area immediately to the Southeast of the “A” Street facilities.
- Between South Street and Van Dorn from 56th to 70th.
- Between 7th Street and 14th Street near Old Cheney Road.
- At several locations in the High Service Level along the boundary between the High Service Level and the Low Service Level between Vine Street and Regent Street from 56th to 70th.

Areas of high pressure such as these are not uncommon in many systems. These areas should continue to be monitored and coordinated with the occurrence of main break rates. No specific recommendations are made in this report to reduce these pressures. However, if these areas become problematic consideration should be given in the future to methods to remediate these areas of high pressures.

5.9.2 WTP Pumping Capacity

The completion of transmission mains currently under construction will create a reduction in the required total discharge head and allow the WTP pumps to operate at a higher flow point on their curves. Therefore, the actual WTP operating capacity would be greater than the theoretical firm capacity. This indicates that the replacement of Pump No.10

with a pump similar to Pump No.11 could be postponed until after 2019. By year 2032 the demand would be high enough that Pump No. 10 would need to be replaced and another high service pump would need to be added.

5.9.3 Belmont Service Level

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 Reservoir has a regulating valve (Valve #102) that is 1/4 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 a motorized control valve (Valve #CV101) that is found along the same pipe that the current partially closed valve is located. 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 Reservoir also has an altitude valve BFV-2, which can be programmed to open/close at various level set points that is not currently used.

The future Southwest Storage Facility modeled in the 2032 analyses should have an overflow elevation of 1400, 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.

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 Reservoir. Under year 2032 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 to 4 mgd under current maximum day conditions. However, the July, 2006 SCADA data used in the calibration efforts showed that this pumping station transferred only an average of 2.0 mgd during these same 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 2032 maximum day demands, the total firm pumping capacity into the Belmont Service Level will be surpassed and a northwestern loop is recommended. Belmont Service Level demands should be monitored in order that the optimal timing for this major improvement can be achieved. Portions of this improvement such as the main on NW 56th Street, detailed in Chapter 8, are included in the phased improvements.

5.9.4 High Service Level

Using the criteria as defined in the previous report, that maximum acceptable storage contribution is based on the top one-third of reservoir volume depleted over a 4-hour duration, hydraulic analyses show that during demand conditions similar to those for the 2019 maximum hour, the maximum acceptable storage contribution in the High Service Level is close to being exceeded. In order to maintain the volume of storage in the High Service Level it will become necessary to transfer a greater amount of water through the PRVs connecting the Southeast Service Level and the High Service Level during peak demands.

The transfer of supply from the Southeast Service Level to the High Service Level serves to maintain storage in the High Service Level and also helps to improve pressures in the southern portion of the High Service Level. Hydraulic analyses show that a new 4.0 MG storage reservoir near Saltillo Road will help to increase localized pressures to the south and provide valuable storage for the High Service Level in an area that would be vulnerable in the case of emergency or peak demands. A control valve constructed near the storage facility would assist in the transfer of water from the Southeast Service Level to the High Service Level and aid in maintaining the storage in this future reservoir.

5.9.5 Northwest Booster District

Based on projected demands in the Northwest Booster District, the Northwest Booster Pumping Station would have sufficient capacity to meet year 2032 maximum day demands of 3.2 mgd (This demand includes service to the existing subdivision southeast of US Highway 34). However, a storage facility in this booster district would provide reliability and floating storage.

5.9.6 Cheney Booster District

Currently the Cheney Booster District operates as a closed system and is supplied by the Cheney Booster Pumping Station. A pressure set-point of 91 psi on the pressure sustaining valve at the station allows the station to deliver a constant hydraulic gradient of about 1,600 feet. The pumps are rated at 175 feet of total dynamic head, but only require about 100 to 120 feet of TDH an there is a throttling valve in the pumping station, so they operate nearer to the rated point. When the Cheney Reservoir comes online with an overflow of 1,580 feet, the pumps will run further out on their curves. Depending on the pumps being operated and the demands in the service level, the Cheney Reservoir may experience relatively faster fill rates. Water age analyses indicate that of this type of operation generally has a positive impact on water age as discussed in the next chapter.

6.0 Water Age Analyses

6.1 General

Hydraulic and Water Quality Modeling was performed for the LWS 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, water age can be quite useful in identifying distribution system deficiencies in terms of water quality.

6.2 Computer Model

One option that the EPA has outlined for the Initial Distribution System Evaluation (IDSE) portion of the Stage 2 Disinfection Byproducts Rule (DBPR), is a system specific study (SSS) involving the use of a detailed, comprehensive, and well-calibrated distribution system model *(Initial Distribution System Evaluation Guidance Manual, USEPA-January 2006)*. Although LWS has received 40/30 certification for the Stage 2 DBPR and is not required to conduct a Standard Monitoring Plan (SMP) or the alternative SSS, a brief discussion is provided in regards to the LWS computer model as it compares to the standards established by the United States Environment Protection Agency (EPA) for purposes of a SSS.

The LWS distribution system model meets all the minimum physical requirements set by the EPA for the use of a hydraulic computer model for the SSS. These basic requirements include all pipes 12-inches and larger, all pipes 8-inches and larger that connect pressure zones, 50 percent pipe representation by length, 75 percent pipe representation by volume, all storage facilities, all control valves affecting the flow of water, and all pumping stations with realistic controls and settings that reflect standard operations.

Two other EPS standards requirement for an acceptable hydraulic computer model used for a SSS consists of specific demand data requirements and verification that the model can simulate actual system performance during the peak month of total trihalomethane (TTHM) formation. Both of these requirements have been satisfied by the LWS computer model assuming that maximum TTHM formation occurs during the October time period. A 24-hour hydraulic calibration was performed for July 19, 2006; and operational verifications were performed for a two week period in January 2007 and a two week period in October 2007. For each of these simulations unique diurnal demands were determined and input to the model to simulate actual demands..

In general, the LWS hydraulic computer model exceeds the criteria set by the EPA and the resulting water age analyses performed for this update provide reliable insight into water quality factors in the LWS distribution system.

6.3 Water Age Scenarios

Several scenarios were performed and the results were analyzed to determine areas of high water age and the impact that modifications to the distribution system and operations might have on residence times. Two unique demand scenarios were developed, and in conjunction with different combinations of operational controls and distribution system inventory, several simulations were performed. The two demand levels, based on January 2007 and October 2007 SCADA data, represent demands typical of minimum month and average month conditions respectively.

Operational verifications were conducted to simulate actual system performance for each of the two demand scenarios. A detailed review and discussion of these verifications is provided in Appendix B – Water Age Operational Validation Memorandum.

The scenarios and alternatives that were developed, simulated, and reviewed in these analyses are described as follows:

- Minimum Month Base (MIN_MONTH_BASE) – Typical minimum month conditions based on January 2007 SCADA using the existing distribution system without improvements or modifications.
- Minimum Month Alternative 2 (MIN_MONTH_ALT2) – Typical minimum month conditions based on January 2007 SCADA data with the addition of the Phase I – Immediate Improvements identified in Chapter 8, which include a main on Northwest 56th Street from O Street to Adams Street, a main on Yankee Hill

Road from 56th Street to 84th Street, and a control valve located on the Low Service Level suction side of the Pioneers Pumping Station.

- Minimum Month Alternative 3 (MIN_MONTH_ALT3) – This scenario is identical to MIN_MONTH_BASE with the exception that the Cheney Reservoir was operated with water levels ranging between 10 feet and 20 feet (effective volume of 1.0 MG) rather than the expected typical range of 20 feet to 40 feet (actual total volume of 2.0 MG).

- Minimum Month Alternative 4 (MIN_MONTH_ALT4) – This scenario is similar to MIN_MONTH_ALT2 but with four modifications to operational conditions. The Vine Street New (to Southeast Service Level) Pumping Station controls were modified to slightly increase pumping time to the Southeast Service Level. The storage volume simulated at the "A" Reservoirs was reduced from 32 MG to 24 MG, representing a hypothetical removal of one of the 8 MG reservoirs on site. The controls maintaining the water level in the Pioneers Reservoir were modified to increase the range of water level fluctuations by four feet. The Cheney Reservoir was operated between the ranges of 10 feet and 34 feet.

- Minimum Month Alternative 5 (MIN_MONTH_ALT5) – This scenario is similar to MIN_MONTH_ALT2 and included two operation modifications. The "A" Pumping Station was set to maintain a constant flow averaging about 10 mgd (5 mgd greater than the other minimum month analyses). The controls maintaining the water level in the Pioneers Reservoir were modified to increase the water level operating range by two feet.

- Average Month Base (AVE_MONTH_BASE) – Average month conditions based on October 2007 SCADA data. The Southeast Reservoir and Pumping Station were out-of-service during this time, but the model was developed to simulate a more typical condition with these facilities online. The existing distribution system without improvements or modifications was used in this simulation.

- Average Month Alt2 (AVE_MONTH_ALT2) – This scenario is identical to AVE_MONTH_BASE with the addition of the Phase I - Immediate Improvements as identified in Chapter 8, which include a main on Northwest 56th Street from O Street to Adams Street, a main on Yankee Hill Road from 56th Street to 84th Street, and a control valve located on the Low Service Level suction side of the Pioneers Pumping Station.

- Minimum Month Operation Parameters with Average Month Demands (MIN_MONTH_OCT_DMND) – This scenario used the diurnal demands developed from the October 2007 data but the operation parameters developed from the January 2007 data as used in the MIN_MONTH_BASE scenario. The existing distribution system without modifications or additions was used in this simulation.

6.4 Observations

6.4.1 General

Simulation results did not always agree with expected results and at times the expectations formulated prior to the analysis seemed to be contradicted by the analysis results. One example of this is illustrated by the occurrence of higher water ages in the Southeast Service Level and portions of the High Service Level under higher demand conditions. These higher calculated water ages were contrary to expected results of lower water ages during periods of high demand. A combination of several factors may contribute to this phenomenon with the most significant being a change in reservoir water level operational ranges. During the October conditions, reservoir levels were generally maintained at higher levels and the fluctuating ranges were slightly reduced when compared to the January conditions.

A review of the water age analyses resulted in the following general observations:

- The fluctuation range of water level in a reservoir has an impact on water ages. Greater ranges of fluctuation generally result in lower water ages.
- The typical maximum and typical minimum water levels maintained in a reservoir has an impact on water ages.
- The amount of volume in the distribution system has an impact on water ages. This volume includes reservoir storage and the volume within the distribution system mains. Greater volumes generally results in higher water ages.
- Even a slight difference in the average flow from a pumping station has an impact on water ages. Evaluation of results is complicated by the fact that some service levels are supplied through multiple pumping stations with differing water age characteristics on the suction side.

- Regardless of operational variations, in almost all cases of the same demand conditions, the overall average distribution system age stays relatively close to the same value. Analyses results indicate that lower water ages in one location of the system are generally counterbalanced by higher water ages in another location. The most notable exception to this is found in review of the results for the MIN_MONTH_ALT5 scenario.

Each water age scenario was simulated for a duration of five weeks (840 hours). Table 6-1 summarizes the average calculated water age for the last two weeks (336 hours) of the simulation for each scenario. Table 6-2 summarizes the average pumping station throughput for the last two weeks for each scenario.

Figures were developed in the review of the water age results and are provided in Appendix C – Water Age Results Figures. Figures 1 and 2 show the average water age over the last two weeks for the base January and base October scenarios. Figures 3 through 9 show the difference in water age between various scenarios.

Table 6-1
Scenario Water Age Results

Reservoir	Average Water Age, hours							
	Minimum Month, MM					Average Month, AM		MM Operations, AM Demand
	BASE	ALT2	ALT3	ALT4	ALT5	BASE	ALT2	
Air Park	474	483	476	461	470	472	511	448
Pioneers	375	347	374	297	288	383	329	331
Northwest	317	347	318	331	370	337	316	295
Southeast	470	514	503	516	361	548	561	493
51st	224	220	225	186	195	271	250	176
Pine Lake	521	545	540	539	455	515	525	515
Northeast	124	122	126	98	120	78	67	82
Vine	372	472	397	426	248	596	591	381
"A"	315	317	327	255	289	338	297	292
Yankee Hill	629	604	639	606	524	663	621	632
Cheney	624	618	642	608	552	672	672	620
Average of All Reservoirs (1)	404	417	415	393	352	443	431	388
Overall System Average Age (2)	322	343	333	316	277	344	341	310

(1) This value represents an average of all storage facilities but disregards the volume of individual facilities.
(2) The Overall System Average Age is the average calculated age for all nodes in the system.

Table 6-2 Scenario Average Flow Rates								
Pumping Station	Average Pumping Station Flow, mgd							
	Minimum Month, MM					Average Month, AM		MM Operations, AM Demand
	BASE	ALT2	ALT3	ALT4	ALT5	BASE	ALT2	
"A" to Low	5.2	5.3	4.8	4.9	10.3	8.2	5.3	6.0
"A" to High	4.9	5.4	4.6	5.6	4.9	8.9	8.9	4.8
Belmont	2.9	2.6	3.0	2.7	2.6	3.7	3.6	3.5
51st Transfer	10.4	10.6	9.4	10.5	15.5	17.0	14.1	11.1
51st Low	17.1	16.2	17.6	15.2	12.0	9.0	9.0	15.9
Southeast	1.2	1.2	1.2	0.2	1.2	1.8	1.8	1.5
Vine to High SL	2.9	2.6	3.0	2.7	2.6	3.7	3.6	3.5
Vine to Southeast SL	1.7	1.7	1.7	2.5	1.8	1.7	1.6	1.8
Northeast	0.8	1.4	0.8	2.5	0.3	4.8	7.8	4.3
Pioneers	1.2	1.5	1.2	1.4	1.4	1.0	1.1	1.3
Total Flow In[(1)]	28.1	28.3	27.9	28.2	27.6	30.9	30.9	31.0

(1) The Total Flow In is the summation of flow entering the system at the A Street location, the Northeast Reservoir, and 51st Street Pump Reservoir.

6.4.2 Pioneers Control Valve

A comparison of the MIN_MONTH_BASE and the MIN_MONTH_ALT2 scenarios indicates that the control valve at the Pioneers Pumping Station causes water ages to decrease by about ten percent at the Pioneers Reservoir. However, this decrease in water ages at the Pioneers Reservoir is accompanied by a slight increase in water age (less than two percent) at Air Park Reservoir and at the Northwest Reservoir (less than ten percent). This increase in water ages in the Belmont Service Level cannot be solely attributed to the operation of the Pioneers Pumping Station because the addition of the main on Northwest 56th Street adds approximately 0.9 MG of volume to the Belmont Service Level.

In the setup of the operation controls for the control valve to control operation of the Pioneers Reservoir, it was recognized that the drawdown for the reservoir is limited by its refill capability. In the MIN_MONTH_ALT4 scenario the controls were set to allow the Pioneers Reservoir water level to drop by four feet more than the typical minimum level. The results of the simulation show that the minimum water level of 38 feet results in an excessively long fill time. For the scenario MIN_MONTH_ALT5 the minimum level was raised to 40 feet (two feet below typical minimum level) and the model results indicate that the reservoir could be adequately refilled. The SCADA data showed that the 40 feet level was reached on one occasion in January 2007 - and the reservoir took almost over 13 hours to fully replenish, barely refilling before morning demands began to pick up.

6.4.3 Addition of Immediate Main Improvements

The Phase I – Immediate Improvements (consisting of the main on NW 56th Street from O Street to Adams and the main on Yankee Hill between 56th Street and 84th Street) were included in all of the alternative analyses following the two base analyses.

A review of the Alternative 2 scenarios indicates that the additional volume attributed to the main on NW 56th St. appears to contribute to slight increases in water ages in the Belmont Service Level.

The impact of the addition of the main on Yankee Hill is difficult to assess. This main increases the distribution system volume by just over 0.5 MG which is a small percentage of the overall volume in the Southeast Service Level. As seen in Table 6-1 the average age in the Yankee Hill Reservoir decreases when this main is placed into service. However, a review of the figures Appendix C indicates that the overall water ages in the Southeast Service Level generally increase. This increase could be caused by several factors including a change in how the system operates when identical control parameters are used with different distribution system inventory. Without the main on Yankee Hill Road, the reservoir operates as if it essentially has only one inlet/outlet pipe. The water that drafts from the reservoir is pushed back in during the fill cycle. This results in lower average water in the Southeast Service Level but higher water in the reservoir. Addition of the main promotes movement of water throughout the Southeast Service Level and reduces the range of water ages in the level.

6.4.4 "A" Pumping and Storage Volume

The water age analyses indicated that a correlation exists between increased pumping at the "A" location and lower water ages throughout the system. MIN_MONTH_ALT5 was specifically developed to review this correlation and the water ages resulting from the analysis were the lowest ages of all the scenarios. The "A" Low Level Pumping Station was set to pump continuously, resulting in about twice the throughput of any other minimum month scenario. In conjunction with increased pumping to the Southeast Service Level from Vine, the scenario resulted in the largest reduction in water ages for the overall system and in all reservoirs except for the Northwest.

The MIN_MONTH_ALT4 scenario illustrates the positive impact that reduced storage volume at “A” has on water ages. Although this scenario also includes several operational modifications, it appears that water ages can be significantly reduced by removing 8.0 MG of storage at “A”. It may be beneficial for LWS to consider taking one of the storage reservoirs at “A” out-of-service during periods of lower demands to reduce water ages, if water quality concerns warrant.

6.4.5 Cheney Service Level and Storage

Some of the highest water ages in the distribution system occur in the Cheney Service Level near the Cheney Reservoir. The scenarios MIN_MONTH_ALT3 and MIN_MONTH_ALT4 reviewed the impact that possible operation changes in the Cheney Booster Pumping Station would have on the water ages in the Cheney Service Level. As indicated by the review of the ages reported for the MIN_MONTH_ALT3 scenario, the operation of the Cheney Reservoir with a fluctuating as a 1.0 MG reservoir range of 10 feet to 20 feet (effective volume of 1.0 MG) increases the water ages in the Cheney Service Level. The cause of this negative impact is likely the result of less water entering and being drawn of by the distribution system resulting in a “sloshing” effect of the same water entering and leaving the reservoir.

6.4.6 Vine Pumping to Southeast

Increasing the throughput of the Vine Pumping Station to the Southeast Service Level was evaluated in the MIN_MONTH_ALT4 scenario. The analysis indicates that increased pumping from the Vine Reservoir with a corresponding decrease in pumping from the Southeast Reservoir increases the average water age in the High Service Level increase decreasing the average water age in the Southeast Service Level. Since the Southeast Service Level typically has some of the highest water ages in the system, the ability to decrease these ages at the expense of the ages in the High Service Level may be beneficial if water quality issues become a concern in the Southeast Service Level.

6.5 Conclusions

Conclusions drawn from the distribution system water age analyses are summarize below:

- Increased pumping at Vine to the Southeast Service Level reduces the water age in the Southeast and Cheney Service Levels with only a marginal increase in water age in the High Service Level.

- Increased pumping at "A" tends to reduce water ages throughout the system.
- The temporary reduction of storage volumes at "A" (and possibly other locations) during periods of lower demand reduces water ages throughout the system.
- The main improvement on Yankee Hill improves the movement of water in the Southeast Service Level and better balances water ages throughout the service level.
- The Pioneers Control Valve moderately improves water ages in the Pioneers Reservoir but has a negligible impact on water ages in the Belmont Service Level.
- The addition of the main on NW 56th Street in the Belmont Service Level increases the storage volume in this service level and minimally increases water age.
- The minimum water level drawdown in the Pioneers Reservoir limited by distribution ability to refill it adequately and appears to be close to 40 feet.

7.0 Water Main Replacement Program

7.1 Existing Mains

The LWS has approximately 1,170 miles of raw and finished mains. LWS reports that pipe materials in the system consist of lined and unlined cast iron (both pit cast and centrifugally cast), lined ductile iron (wrapped and unwrapped), asbestos cement (also referred to as Transite pipe), pre-stressed concrete cylinder, steel, and polyvinyl chloride (PVC). PVC, which has been used since 1994, is only allowed on 6 to 12 inch mains. LWS reported the historical materials generally used for small-diameter (16-inch and smaller) distribution mains as summarized below.

- 1890 to Early 1960s: Cast Iron Pipe
- Early 1960s to 1980 Ductile Iron Pipe
- 1980 to Present Poly Vinyl Chloride Pipe and Ductile Iron Pipe

Service connection materials are copper or galvanized steel. Since the late 1970's, all ductile iron piping has been wrapped with polyethylene encasement (poly-wrap) to reduce corrosion. For several years prior, soils testing was required and only those mains located in corrosive soils were required to be poly-wrapped. The LWS then determined that it would be more cost-effective to poly-wrap all ductile iron piping instead of performing the soil corrosivity tests. The LWS has required wrapping of all ductile iron pipe with a double layer of polyethylene encasement since 1998.

The LWS maintains a spreadsheet that provides an annual total of the length of pipe maintained by LWS. The spreadsheet includes detailed information for all construction projects showing the diameter and length of new main construction since 1976, and the diameter and length of abandoned mains since 1981. The total length of pipe for 1976 is based on previous historical accounting and it is not clear whether raw water mains and transmission mains from the water treatment plant to the City of Lincoln are correctly accounted for. Further, it is recognized by LWS that there may be inaccuracies in the spreadsheet because abandoned mains may not be sufficiently accounted for in past years, resulting in an over-estimation of the actual length of pipes maintained by LWS.

Detailed mapping of the water distribution system is maintained on an electronic line drawing (Microstation line drawing) called the "water foremans plats". In 2004, the line drawing was used as the basis for creating the geodatabase of the existing distribution system. Water main, hydrant, and valve elements were captured from the Microstation drawing during the geodatabase creation. Text annotation on the drawing was used to provide the initial diameter attribution in the geodatabase. However, this process resulted in many of the pipe elements in the geodatabase not having an assigned diameter information (attribute) because there were many more individual line elements in the drawing than there were text elements. The LWS GIS section is in the process of reviewing mapping and records of the distribution system to verify and complete the main attribution for diameter, material, and installation year. As of the writing of this report, about 50 percent of the GIS had been thoroughly checked and updated based on this review. The current geodatabase cannot be used in its present state to readily quantify the amount and size of mains in the distribution system.

As described in Chapter 5, the distribution system hydraulic model includes all gridded, or looped, mains of 4-inch diameter and larger. The model created for the *2002 Facilities Master Plan* was created from the Microstation line drawing with an extensive quality control process to verify diameters and line elements. Hydraulic calibration of the model further verified its accuracy. The LWS provided a copy of the distribution system geodatabase in June 2007, which was used to update the 2002 model. New mains in the geodatabase were added, and abandoned mains were removed, to create the 2007 model. The 2007 distribution system model contains all finished water mains (except for a small amount of small-diameter unlooped mains) including the transmission system piping from the water treatment plant to the City of Lincoln. The 2007 model provides a solid basis for quantifying the amount and size of mains in the distribution system.

A hydraulic model of the raw water piping in the wellfield supplying the water treatment plant was created for a separate project. The wellfield piping model provides a solid basis for quantifying the amount and size of mains in the raw water wellfield piping.

The total length of buried mains maintained by the LWS was determined from the information in the water distribution system model and the water treatment plant wellfield model, and is summarized in Table 7-1.

Table 7-1
Main Lengths by Service and Diameter

Diameter	Finished Water Mains (mi) Transmission Level [(1)]	Distribution Levels [(2)]	Total Finished Water	Raw Water Mains [(3)] (mi)	Total Water Mains (mi)
4	--	65.6	65.6	--	65.6
6	--	497.8	497.8	--	497.8
8	--	73.9	73.9	--	73.9
10	--	11.7	11.7	--	11.7
12	--	144.0	144.0	0.6	144.6
14	--	0.1	0.1	--	0.1
16	--	106.4	106.4	1.1	107.5
18	--	0.1	0.1	0.0	0.1
20	0.3	7.6	8.0	1.2	9.2
24	2.7	45.1	47.8	1.6	49.4
30	0.04	13.0	13.1	1.4	14.4
36	28.8	10.4	39.2	6.7	45.9
42	--	0.1	0.1	0.1	0.2
48	21.0	12.5	33.5	1.8	35.3
54	7.9	3.5	11.4	3.0	14.4
Total	**60.9**	**991.7**	**1052.6**	**17.6**	**1,070.2**[(4)]

(1) Transmission level mains consist of all mains required to deliver finished water to the City of Lincoln – including all mains which supply the Northeast Reservoir, 51st Reservoir and the "A" Reservoirs, plus the transfer line from the Vine Reservoir to the "A" Reservoirs.
(2) Distribution level mains consist of all mains in the Low, Belmont, High, Southeast, Northwest, and Cheney Service Levels.
(3) Raw water mains shown here include only the mains in the wellfield at the WTP. Raw water mains for the Local Wellfield at the "A" Reservoirs facility are not included in this table. It is estimated that there are less than 4 miles of raw water mains comprising the Local Wellfield.
(4) Total water main length shown here, based on information contained in detailed hydraulic models, is about 140 miles less than the 1,211 miles of main reported in the LWS summary spreadsheet for 2007.

LWS allows service connections on mains up to and including 16-inches in diameter as a general rule. Approximately 899 miles of water mains are small-diameter finished water distribution mains, 16-inch diameter and smaller, comprising about 85% of all finished water mains.

Table 7-2 shows the length of new water main construction, mains abandoned, and annual finished water main length from 1976 to 2007. Over the 32 year time period from 1976 through 2007, a total of about 508 miles of new piping was constructed, while about 37 miles of existing piping was abandoned. On average, almost 15 miles of length was added to the system annually. Of the approximately 1,050 miles of finished water main in the existing system, almost 50 percent (48.2%) of it has been constructed in the past 32 years.

Table 7-2
Annual Distribution System Expansion

Fiscal Year(1)	Finished Water Main Changes (feet)(2) Total Mains Added	Large Dia. (>16") Added	Total Mains Abandoned	Total Finished Water Length (mi)(3)	Total Small Diameter Length (mi)
1976	1,300	0	NA	601.7	491.6
1977	113,843	10,600	NA	601.9	491.8
1978	155,712	2,346	NA	623.5	511.4
1979	102,892	16,846	NA	653.0	540.4
1980	122,342	19,012	NA	672.5	556.7
1981	72,205	19,012	8,164	695.6	576.3
1982	15,949	13,321	445	707.8	584.8
1983	6,422	0	1,900	710.7	585.2
1984	25,666	5,400	0	711.5	586.1
1985	29,108	891	0	716.4	589.9
1986	55,516	11,217	2,260	721.9	595.3
1987	16,790	0	695	732.0	603.2
1988	29,979	3,662	2,928	735.1	606.3
1989	56,172	120	2,353	740.2	610.7
1990	43,104	1,275	1,500	750.4	620.9
1991	37,820	0	9,285	758.2	628.5
1992	62,217	0	0	763.7	633.9
1993	82,036	8,561	5,089	775.4	645.7
1994	77,636	0	4,612	790.0	658.7
1995	53,439	0	5,160	803.8	672.5
1996	78,881	7,891	9,329	813.0	681.6
1997	67,415	2,220	0	826.2	693.3
1998	111,515	15,193	4,662	838.9	705.7
1999	125,235	957	19,570	859.2	723.0
2000	171,901	8,440	6,435	879.2	742.8
2001	96,377	12,043	14,357	910.5	772.6
2002	117,481	940	8,875	926.0	785.8
2003	110,856	169	7,800	946.6	806.2
2004	166,720	20,569	8,968	966.1	825.7
2005	168,843	33,127	3,683	996.0	851.7
2006	134,344	13,692	707	1027.3	876.7
2007	170,710	42,381	68,246	1052.6	899.4
Total (ft)	**2,680,425**	**269,886**	**197,023**		
Total (mi)	**507.7**	**51.1**	**37.3**		

(1) Fiscal year (FY) is from September of previous year to August of current year. For example, FY 2007 is from September 2006 to August 2007.

(2) Length of annual mains constructed and abandoned is from spreadsheet provided by LWS titled “Current LWS Watermains”.

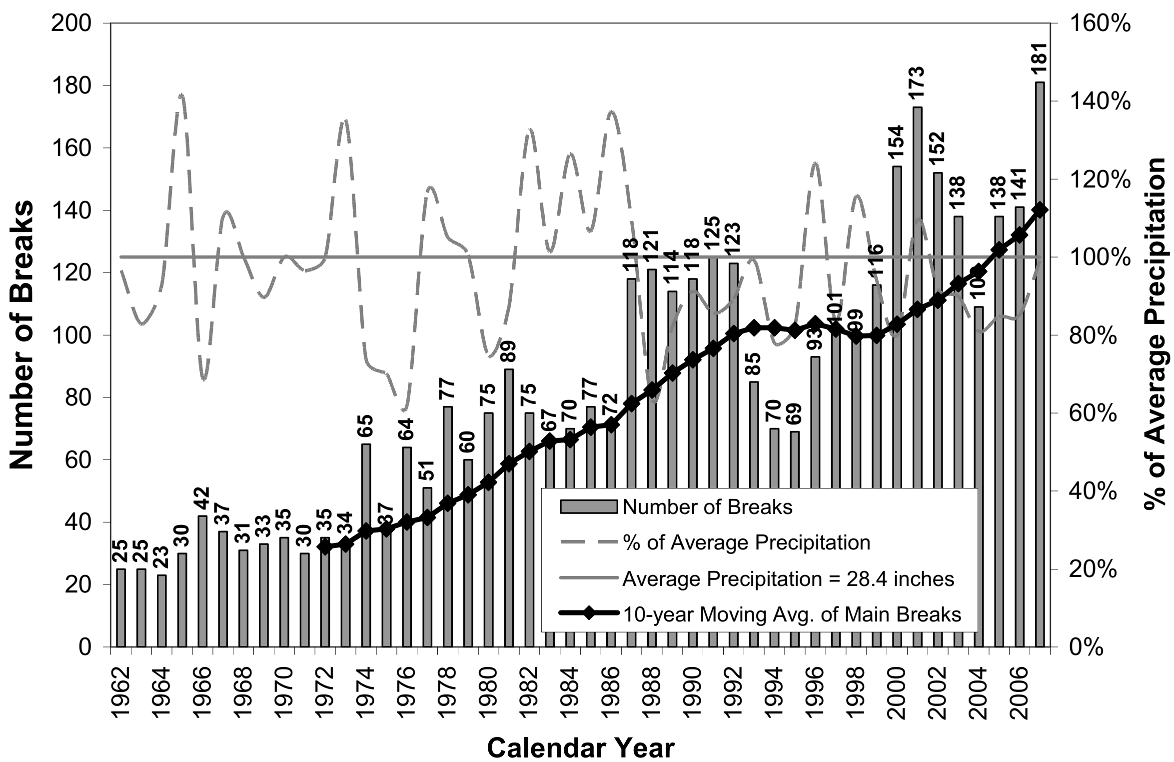
(3) Annual total length of system is based on 2007 total length of distribution mains of 1052.6 as determined from detailed hydraulic model and reported in Table 7-1.

7.2 Main Breaks

7.2.1 Historical Main Breaks

LWS provided a summary spreadsheet of historical main breaks by year from 1962 to 2007. LWS reports that this summary data since 1991 is based on the number of work orders issued for main break repairs, as documented in their Hansen Work Order Management System (WOMS). Main break history prior to 1991 is based on previous historical paper records. Figure 7-1 shows the number of water main breaks each year from 1962 to 2007 compared to annual precipitation.

Figure 7-1 Historical Main Breaks and Precipitation



It is difficult to draw a direct correlation between the relatively dry years and an increase in water main breaks from Figure 7-1. The *2002 Facilities Master Plan* showed that the 85 percent of the main breaks are occurring on small-diameter (6-inch and 4-inch) cast iron pipes. This observation is similar to other utilities in the United States.

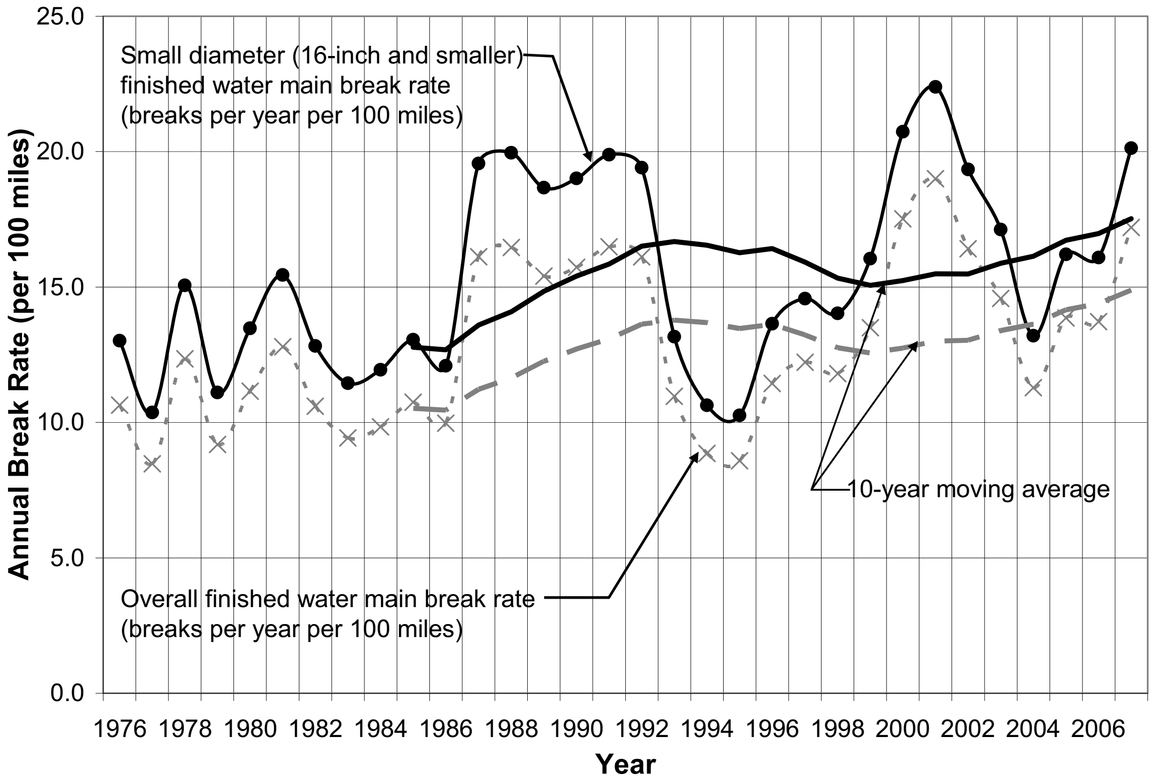
The number of breaks per year is not necessarily a good indication of system condition. A better indication is the number of main breaks per unit length. The historical record of length of main in system, as reported in Table 7-2, was used in conjunction with the number of breaks per year to calculate that annual main break rate for LWS. Table 7-3 and Figure 7-2 show the annual main break rate from 1976 to 2007.

Table 7-3
Historical Main Break Rates

Year	Number of Breaks	Total Finished Water Mains		Small Diameter (<=16") Finished Water Mains	
		Length (mi)(1)	Break Rate per 100 miles	Length (mi)(1)	Break Rate per 100 miles
1976	64	602	10.6	492	13.0
1977	51	602	8.5	492	10.4
1978	77	623	12.4	511	15.1
1979	60	653	9.2	540	11.1
1980	75	672	11.2	557	13.5
1981	89	696	12.8	576	15.4
1982	75	708	10.6	585	12.8
1983	67	711	9.4	585	11.4
1984	70	712	9.8	586	11.9
1985	77	716	10.7	590	13.1
1986	72	722	10.0	595	12.1
1987	118	732	16.1	603	19.6
1988	121	735	16.5	606	20.0
1989	114	740	15.4	611	18.7
1990	118	750	15.7	621	19.0
1991	125	758	16.5	629	19.9
1992	123	764	16.1	634	19.4
1993	85	775	11.0	646	13.2
1994	70	790	8.9	659	10.6
1995	69	804	8.6	672	10.3
1996	93	813	11.4	682	13.6
1997	101	826	12.2	693	14.6
1998	99	839	11.8	706	14.0
1999	116	859	13.5	723	16.0
2000	154	879	17.5	743	20.7
2001	173	911	19.0	773	22.4
2002	152	926	16.4	786	19.3
2003	138	947	14.6	806	17.1
2004	109	966	11.3	826	13.2
2005	138	996	13.9	852	16.2
2006	141	1,027	13.7	877	16.1
2007	181	1,053	17.2	899	20.1

(1) Annual total length of system is based on 2007 total length of distribution mains of 1052.6 as determined from detailed hydraulic and reported in Table 7-1.

Figure 7-2 Historical Main Break Rates



The 10-year average main break rate has slowly risen over the years. Figure 7-1 shows that the 10-year average for the overall system, considering all finished water mains, is about 15. If only the small diameter mains (mains 16-inch diameter and smaller) are considered the break rate is about 17. This level of main break activity is below the rates many water utilities in the US experience. The 1995 AWWARF report Distribution System Performance Evaluation states that “Analysis of historical main break data of various water systems shows that a “reasonable goal" for main breaks for a water system in North America is 25 to 30 main breaks per 100 miles per year.” The lower rate in Lincoln should be considered in light of the relatively newer age of the distribution system. As reported earlier, almost half of the LWS finished water mains have been constructed in the past 32 years.

LWS staff reports that the primary suspected causes of water main breaks are settlement beneath the pipe, corrosion resulting from the impact of aggressive or corrosive soils on ductile iron and to a lesser extent cast iron pipes, and pipe stresses resulting from differential pressures in the distribution system and heaving soils because of extremely cold temperatures. They reported the following observations of pipe failures in the distribution system:

- Cast Iron and ductile iron are the most common pipe materials related to failures and are the most common material for small diameter pipes. A high rate of ductile iron pipe failures appear to occur in highly corrosive soil or where the poly-wrap has been damaged.
- PVC pipe has a low failure rate. Most failures occur when the pipe is being tapped or when it is damaged from another excavation or drilling operation.
- Pit cast has been uncovered and is in fairly good shape
- Asbestos cement pipe has a higher rate of fitting failures than other materials.

7.2.2 Main Break Geodatabase and CMMS

LWS has recently created a geodatabase that allows for spatial mapping of main break locations. The geodatabase was created by LWS based on information contained in two sources.

The first source of data for the geodatabase is an Access database that contains detailed information for many of the main breaks that occurred from 1984 to November 2006. The data in the database is taken from a form that the work crews fill out upon completion of a main repair. The form is not required, nor completed, for all main breaks, but provides LWS a good partial history of historical main break causes. While the forms are still filled out for many of the main breaks, the information has not been entered into the Access database since November 2006.

The Hansen computerized maintenance management system (CMMS) provides the second source of data used in the main break geodatabase. In November 2006 LWS began entering some of the collected main break data into the CMMS, however, only the following information is entered into the CMMS.

- Description of break (circumferential crack, longitudinal break, pitting).
- Location of break (top, bottom, etc.).
- Type of corrosion.
- Extreme temperatures recorded in past week.

- Suspected cause of break (pressure surge, frost, shear).

Although this additional limited data has only been entered recently into the CMMS, the CMMS has been is use since 1991 for work order management, and therefore provides an accurate count of work orders issued for main repairs since 1991. The CMMS is the source that LWS currently uses for determining the historical number of main breaks that occur in the distribution system.

Soil information was obtained from the US Department of Agriculture, National Resource Conservation Service (NRCS). A soil corrosion risk map was prepared to compare high-risk soils to water main break locations as provided in the LWS geodatabase. This information is summarized on Figure 7-3 at the end of this chapter. It is difficult to assess the impact of potentially corrosive soils based on a simple review of Figure 7-3. Other factors such as pipe material and age need to be considered in evaluating the impact of corrosive soils. Additional detailed statistical analysis, after relating the main breaks to pipes would need to be conducted to provide an indication of the relative impact of corrosive soils on the incidence of main breaks.

7.3 Main Replacement Program

The LWS updates the list of main replacements approximately every other year and identifies approximately 5 years of projects. Over the past 10 years, the replacement rate has averaged about 2.8 miles per year, with a low of only 707 feet in fiscal year 2006, and a high of 42,400 feet (8.0 miles) in fiscal year 2007. About 26,800 feet (5.1 miles) are budgeted for replacement in fiscal year 2008, and this rate is projected to remain relatively constant over the next 6 years. The 5.1 miles of main replacement represents about ½ of 1 percent (0.5%) of the total length of small-diameter (16-inch and less) finished water transmission mains. At a replacement rate of 5 miles per year, it would take nearly 180 years to replace all of the small-diameter finished water mains.

The pipeline replacement program budget has increased gradually from $500,000 in 1992 to its current level of $2.75 million for fiscal year 2007/2008. The budget is planned to increase to $3.2 million by fiscal year 2012/2013. The pipeline replacement program accounts for nearly 15 percent of total 2007/2008 capital improvements budget of $19.4 million.

The LWS develops the planned list of pipeline replacement projects on an annual basis. The following general considerations are reported to be used as primary considerations for developing the program:

- Locations with a high frequency of breaks
- Opportunity projects resulting from coordination with street repaving and construction projects, and coordination with sanitary sewer construction projects
- Locations with users whose public or commercial impact would be severely affected by loss of water (schools, hospitals, restaurants, hair salons, etc.)
- Potential problem areas located within the right-of-way of public roads
- Areas with known highly corrosive soils

7.4 Routine Maintenance Activities and Programs

7.4.1 General

The LWS has 37 full time staff in the operations and maintenance section who are responsible for the operation, repair and maintenance the water distribution mains 16-inch diameter and smaller. This staff includes management, supervisory and field crews.

Information on the number of appurtenances is maintained in the Hansen CMMS. LWS provided the following summary inventory of distribution system assets maintained by the section as of January 2008:

- 10,130 fire hydrants.
- 23,532 valves including hydrant branch valves.
- 1,170 miles of water main (includes distribution and transmission within Lincoln).
- 5,176 backflow devices (privately owned but testing is managed by Lincoln Water System).
- 80,518 water service lines (privately owned but inspections, leak control and locating is managed by Lincoln Water System).
- 80,078 meters.

Maintenance and repair activities conducted by the distribution system section include the following:

- Small-diameter (16-inch and smaller) water main break and leak repairs.
- Localized flushing in response to water quality complaints.
- Limited dedicated flushing in areas prone to water quality issues through the use of automatic flushers.
- Contractor assistance and shutdowns for water main replacement projects.
- Final inspections and follow-up on new water main and replacement water main construction.
- Response and follow-up on leaks, maintenance and repairs on customer owned water services.
- Inspections of new water service connections and meter installations.
- Annual fire hydrant inspections and follow-up repairs.
- Inspections of small-diameter (16-inch and smaller) valves and follow-up repairs.
- Meter testing, maintenance and replacements.
- Cross connection control program including records management and notification to customers regarding annual backflow testing.
- General customer service response.
- Emergency response.

7.4.2 Routine Inspection and Flushing Programs

LWS reports that routine inspections programs include the inspection of every valve in the distribution system approximately every five years and of every hydrant in the system on an annual basis. There is no special inspection program for large-diameter or critical valves.

The hydrant inspection program consists of opening the hydrant until a small amount of flow exits the hydrant. The crews verify that the valve is operable, and that the hydrant drains adequately. If noticeable discolored water is evident, and outside temperatures allow, the crews open the hydrant more fully and flush until the water clears. In addition to this "random" conventional flushing program that occurs because of the hydrant inspection program, hydrant flushing is also initiated in direct response to reported water quality complaints. There is no "uni-directional" flushing program.

The distribution section generally has one dedicated crew consisting of two to three people responsible for field inspections and exercising programs for valves and hydrants. It was reported that in 2007, the routine inspection crew became so involved in assistance with contractor shutdown activities that routine inspection and flushing programs were not completed to the extent desired or planned. It was report that only about 80 percent of the hydrants were inspected in 2007.

Work orders are generated from the CMMS for the separate valve and hydrant inspection / exercising programs, and each of the programs are tracked in the CMMS. Group work orders are issued for hydrant inspections, and separate group work orders are issued for valve inspections. Group work orders are generated by plat/foremans map. When the crew has completed the inspection of hydrants or valves in an area covered by the group work order, a slip of paper that indicates the work has been completed, is provided to an entry clerk. The clerk enters the information into Hansen CMMS, and all valves (or hydrants) in that plat are tagged as having been inspected on that date. There is no method to specifically identify that all valves (or hydrants) have actually been inspected.

When a valve (or hydrant) is identified as having a failure that cannot be fixed immediately with the inspection crew, the inspection crew notes this and provides a list of valves (or hydrants) that require additional repairs. A separate work order is then created for these valves (or hydrants). LWS reports that there most common reported valve failures are broken gates or bolt corrosion.

7.5 Literature Review

There is a very large body of knowledge that is relevant to the practice of assessing the condition of distribution pipelines and the development of plans and budgets for optimizing the long-term performance of these assets. Recent attention to this practice has been heightened by GASB 34 and increased attention to the aging infrastructure and renewal funding gap issues. However, water utilities have dealt with the effects of aging distribution pipelines for many years, and this is reflected in pertinent literature articles dating back to at least the late 1970s and early 1980s.

What has changed recently is the formal recognition that issues related to assessing and maintaining distribution pipelines fall within the category of "asset management". As a result, more recent articles focus on the development and deployment of systematic planning approaches. These approaches incorporate many different facets of the pipeline condition assessment process.

A variety of information sources were reviewed pertaining to the different components of a pipeline condition assessment. This information has been classified into five main categories of reference sources as described in the following sections.

7.5.1 Failure Prediction

The causes and rate of deterioration of distribution pipelines is a key consideration in conducting a pipeline condition assessment. Much of the literature utilizes historical main break data to derive predictions for future failure rates. Historical utility data relating main breakage and replacement rates to pipeline characteristics and environmental factors is often available, although the quantity and consistency of the data varies both between and within individual water systems.

There is a fair amount of literature relating how utilities, researchers and practitioners develop and analyze patterns and trends between pipeline characteristics (age, material, diameter) and environmental influences (i.e., soil conditions), and the experienced rate of deterioration. Statistics and probability functions are used to develop probabilistic forecasts for main break rates and estimated life expectancies. These predictions are usually based on homogeneous groupings of pipe and environmental characteristics.

Other literature demonstrates the use of similar sampling and statistical methods to derive pipe failure probabilities based on pipe condition assessment (i.e., inspection, data). Inspection techniques are in themselves an important component of a pipeline condition assessment program and are discussed with respect to the review of literature pertaining to the inspection, maintenance, and renewal technology category (See Assessment and Renewal Technologies below).

Regardless of whether the failure rates are derived from historical main break or pipeline inspection data, they can be used with the costs of maintenance, renewal, failure, and replacement to conduct economic evaluations in support of pipe rehabilitation and replacement strategies.

7.5.2 Economic Analysis

The economic analysis category includes the literature on three related parts of a pipeline condition assessment:

- Break-even/life cycle cost analysis.
- Macro-analysis for budget forecasting.
- Risk management.

There are a significant number of literature articles dealing with the use of cost-based methods to assist with decisions related to the management of water distribution pipelines. A large majority of these articles focus on the use of break-even analysis or life cycle costing to identify the optimal replacement time for a pipeline or a homogeneous group of pipes. Approaches such as these may also account for the use of proactive rehabilitation and replacement strategies to improve service and minimize total costs.

The total present value cost of a distribution pipe is represented by the sum of the discounted failure/repair, inspection, and rehabilitation/replacement costs. It represents the total amount of money that must be set aside today to finance the pipeline's continual repair and/or eventual replacement. Many articles recognize that the end of life of a water main corresponds to its economic life, or the optimal time for replacing the main.

Numerous articles note the importance of identifying and attempting to quantify the indirect and social/goodwill costs associated with main breaks. Consideration of these factors is necessary to level the playing field between reactive and proactive distribution infrastructure management. Health risks, higher treatment costs for disinfection and corrosion control, flushing, and customer complaints are other factors that help to establish the desired service level for the distribution system, and the literature notes that this will also affect the optimal replacement timing of distribution mains.

The concept of macro-analysis for distribution infrastructure planning was pioneered in the mid- to late 1990s and includes the KANEW and NESSIE models. These models use survival functions developed from the analysis of breakage and/or replacement data to forecast main replacement needs on a system-wide basis in terms of the miles of main per year or an average annual budget in dollars. The literature points out that the models are limited to long-range planning and do not provide location-specific replacement and rehabilitation information. Nevertheless, they are often developed in conjunction with prioritized implementation plans for individual projects and used to establish a consistent level of infrastructure investment over any desired budget period.

Failure management is necessary for the economical operation of distribution pipelines. The idea is to manage the failures consistent with the optimal timing for renewal. There are a number of articles, however, that point out that failure prevention may be a more appropriate approach for large-diameter mains because of the potential for very large failure consequences. In these cases, additional measures are recommended to attempt to anticipate the failures and prevent their occurrence.

7.5.3 Implementation Planning (Micro-Analysis)

One of the primary objectives of most pipeline condition assessment programs is the development of a plan to enable a utility to proactively implement projects that will lower the long-term cost of operating its distribution system, while maintaining or increasing the level of service to its customers. Accordingly, literature for this category incorporate topics related to capital investment planning, project prioritization, and methodologies for identifying and applying criticality factors to projects.

Implementation planning involves micro-analysis to make project-level recommendations. A number of literature sources describe approaches for extending budget forecasting to define specific actions and their associated costs, which become part of a water utility's capital investment program (CIP).

While the CIP plan should be consistent with the results of the economic analyses indicating the recommended timing of renewal or replacement for different pipe categories, the priority for individual pipeline projects also depends on criticality factors that do not necessarily relate to the condition of a pipeline or its predicted rate of deterioration. These may include, for example, coordination with utilities/public works projects, roadwork, project size, and sensitive customers (hospitals, schools, etc.).

A number of literature articles outline the use of a weighted-score approach for ranking projects based on condition-related and non-condition criticality factors. A similar approach can be used to conduct a risk analysis for large-diameter pipelines. Instead of a prioritized project list, the risk analysis would provide a basis for prioritizing further actions, including pipeline testing and inspection.

7.5.4 Assessment and Renewal Technologies

There are a significant number of literature articles that address the testing/inspection techniques and rehabilitation methods for water distribution pipelines. Knowledge of available inspection and rehabilitation technologies is an essential component of a pipeline condition assessment program. Research is ongoing both within and outside the water industry to develop new technologies that can improve the accuracy, reliability, and cost-effectiveness of pipeline inspection and rehabilitation. Much of the research is focused on non-destructive examination (NDE) methods and trenchless rehabilitation.

Pipeline inspection activities are performed at various stages of the condition assessment process. Initial inspections could be conducted to establish baseline data regarding the condition of representative pipelines throughout the system. Follow-up testing is often recommended as a means of confirming the need and urgency for renewing or replacing individual pipelines, especially in cases where renewal of larger, critical pipelines would be at a considerable expense to the utility.

There are a number of different kinds of inspection techniques that are employed within the industry. Some are capable of discerning pitting, graphitization, cracks, leaks, or breaks on the pipes or at joints between pipe sections, while others use measurements of surrounding soil parameters such as moisture content, pH, electrical resistivity, and stray current potential to gauge the potential for corrosion. Some require samples to be analyzed in a laboratory, while the majority are conducted in-situ, either with the pipeline in service or with it removed from service.

Literature articles regarding pipeline rehabilitation generally focus on methodologies that can be classified into one of three general categories: non-structural, semi-structural, and fully structural. The majority of these methodologies are considered trenchless in that they can be accomplished by exposing only relatively small sections to gain access to the pipeline. Conventional removal and replacement of the pipeline (or abandonment in place) by the open-cut method is another option. Many of the trenchless options utilize various methods for lining the existing pipeline with a coating or a smaller-diameter liner pipe.

Most articles focus on the applicability of the various methods to specific pipeline rehabilitation circumstances or compare the relative advantages and disadvantages, including cost. For example, the cost of surface restoration, traffic impacts, impacts on other utilities, and overall accessibility are key considerations in deciding between conventional and trenchless rehabilitation alternatives. The adequacy of hydraulic capacity, existence of water quality problem or leakage, and structural integrity of pipelines are noted as being the key factors for selecting the appropriate rehabilitation technology category.

7.5.5 Planning Tools and Best Practices

The use of modern planning tools and best practices for managing distribution assets is a common theme among many literature articles. The most prevalent tools include the use of geographical information systems (GIS) and relational databases. The use of GIS and database programs is well-accepted in the industry, as evidenced by a number of commercially available software packages. The various applications for their use in solving problems related to pipeline condition assessment is of interest. GIS and databases serve an essential role in identifying and quantifying the temporal and spatial cause and effect relationships in the data, as well as providing a means to efficiently apply the relationships to the projection of future conditions and corresponding rehabilitation plans.

The use of computerized hydraulic models and related water quality modeling software is also well-represented in the literature and have applications most closely related to defining appropriate rehabilitation technologies and helping to establish project priorities during implementation planning.

Pipeline condition assessment projects can involve a number of engineering, statistical, logical and financial computations. Due to the systematic nature of the planning process for these projects, a number of literature articles describe the development and use of various customized programs to improve efficiency. A few literature articles describe the use of custom applications that were developed to optimize solutions taking into account water quality, hydraulics, and cost, as well as pipeline renewal needs.

In addition to the benchmarks to be discussed later is this chapter, other indicators that are useful for assessing distribution system performance are based on water quality and hydraulics. The number of customer complaints, particularly those that can be attributed to corrosion-related problems, is important information that is frequently discussed in the literature. Comparisons between benchmarks based on the number of customer complaints and those associated with main breaks and leakage may be helpful in defining appropriate renewal strategies. Benchmarks associated with distribution hydraulics, for example, the number of fire hydrants with insufficient flows, can be viewed to gain insights regarding the relative importance of resizing versus renewal.

7.6 Key Distribution Benchmarking

This section provides a comparison of LWS with other utilities for key distribution system benchmarks. In reviewing these comparisons readers are cautioned that the benchmarks represent only one snapshot in time. Therefore, it is not intended as a singular gauge for LWS's overall distribution system performance because adjustments in practices, and aging of the system, can result in changes to the measured performance parameters over time.

Industry benchmarks are typically used to assess the overall condition of a system or group of assets within a system (i.e., distribution pipelines). Due to differences between utilities, however, no single performance indicator should be used as the basis for change. Even multiple indicators should be considered relative to the local factors that may be responsible for a deviation from industry values.

Two water industry reports published within the last 5 years are referenced in the next section. They provide a background on the magnitude and timing of replacement needs associated with deteriorating pipeline based on data from 20 utilities.

Main break rates, leakage rates, renewal rates, and costs are some of the more common performance indicators used to benchmark distribution system performance. Representative water industry values for these indicators are available through the American Water Works Association's (AWWA) Water:/stats database. This database was used to make comparisons between LWS and peer utilities. The findings from this review are summarized in Section 3.2.

7.6.1 Industry Perspective

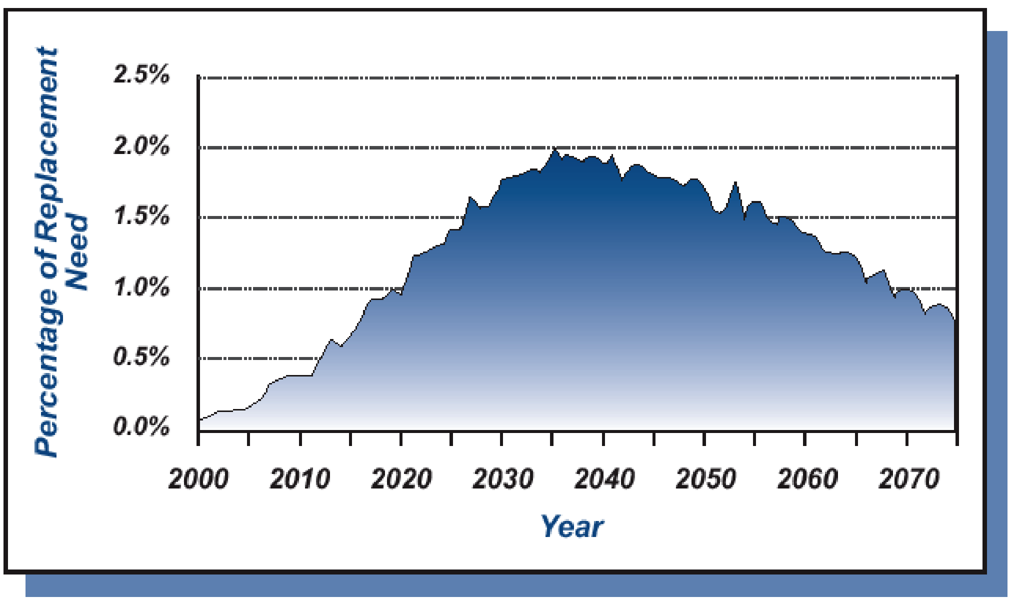
AWWA's *2001 report Dawn of the Replacement Era* reports on the findings of a study of 20 utilities, which was conducted to assess the impact of deteriorating infrastructure on future reinvestment needs. Pipeline replacement and repair expenditures were forecasted for each of the utilities in the study. The following are among the key findings:

- Historical main replacement funding has been adequate; current main breakage rates, overall, are within the range associated with best management practices.
- The largest increase in pipeline replacement needs will occur over the next three to four decades, with the peak occurring between 2030 and 2040 for most utilities.

- On average, required expenditures for pipeline repair and replacement will be about three and three and a half times, respectively, the current spending levels.

The 2002 United States Environmental Protection Agency (EPA) report *The Clean Water and Drinking Water Infrastructure Gap Analysis* was prepared to identify whether a funding gap will develop between projected investment needs for pipeline replacement and projected spending. The Gap Analysis used data from the May 2001 report by AWWA *Dawn of a Replacement Era* as the basis for the reinvestment needs for water infrastructure. For the Gap Analysis, a simple aging model was developed to predict when pipes would need to be replaced. It was assumed that pipes installed before 1910 last an average of 120 years. Pipes installed from 1911 to 1945 were assumed to last an average of 100 years. Pipes installed after 1945 were assumed to last an average of 75 years. Figure 7-4 shows the aggregate forecasted investment needs for the 20 utilities between the Year 2000 and 2075.

Figure 7-4 Forecasted Pipeline Reinvestment for 20 US Utilities



Industry benchmarks are difficult to establish because of differences in the pipeline inventory and rate of deterioration between individual utilities. Nevertheless, two of the most widely-used industry guidelines are a utility's main replacement rate and its experienced main break rate. The two guidelines are related. As the experienced break rate increases, utilities will find it more economical to replace mains experiencing high break rates rather than incur ever-increasing repair costs. The recommended time for replacement represents the utility's estimate of the economic life of a pipeline.

Data in Table 7-4 were adapted from the 2001 AWWA Report for a few selected utilities. These data illustrate the variation in the current and projected reinvestment needs for individual utilities. Although the peak reinvestment year is consistent among all utilities (about 2030), there is a significant difference in the required rate of increased reinvestment for pipeline replacement.

Table 7-4
Reported Replacement Needs for Select Utilities

Utility	Peak Replacement Year	% of Replacement Need		Replacement Cycle (Years)		Annual Average Increase
		2000	Peak Year	2000	Peak Year	
Austin, TX	2030	0.1	1.1	940	90	8.3%
Boston, MA	2030	0.1	0.6	770	180	5.0%
Louisville, KY	2025	0.3	1.5	310	70	6.2%
New Rochelle, NY	2030	0.6	1.5	180	70	3.4%
Philadelphia, PA	2030	0.6	0.8	180	130	1.1%
Portland, OR	2030	0.2	1.0	420	100	4.8%
Seattle, WA	2030	0.2	1.1	570	100	6.2%
Average	**2029**	**0.3**	**1.1**	**487**	**106**	**5.0%**
20-Utilities Average	**2035**	**0.1**	**2.0**	**1000**	**50**	**8.9%**

Source: AWWA 2001 Dawn of the Replacement Era

For example, whereas the City of Philadelphia is currently replacing about 0.6 percent of its distribution pipelines, the City of Austin is replacing about 0.1 percent. This does not necessarily mean that Philadelphia is currently performing better than Austin with respect to pipeline replacement. However, Austin is facing a much steeper increase in replacement expenditures between the Year 2000 and 2030 than Philadelphia. If Austin is still replacing only 0.1 percent of its system in say, 10 years, then this may be an indication that its main replacement program is falling behind. At that point, the City of Austin may have also experienced a significant increase in its main break rate.

7.6.2 LWS Benchmarking Comparisons

The American Water Works Association (AWWA) conducts periodic surveys to compile information about the financial and operating characteristics of member water utilities. The results of the surveys are available in the form of a Microsoft Access database. The 2002 survey results for distribution system characteristics is incorporated into AWWA's Water:/stats 2002 Distribution Survey (Water:/stats). This database contains water distribution data from 337 small, medium, and large drinking water utilities surveyed in 2002 and 2003 on a wide array of potable water distribution characteristics, including :

- Utility Information.
- Pipe Material.
- Valves.
- Fire Hydrants and Flushing.
- Finished Water Storage Facilities.
- Water Conveyance.
- Corrosion Control.
- Customer Metering.
- Customer Service Lines.
- Water Supply Auditing.
- Leakage Management.
- Infrastructure.

The Water:/stats database was used to develop groups for comparison with LWS and to derive benchmarks for assessing relative performance with respect to distribution main breaks, and valve and hydrant inspection programs.

7.6.2.1 Main Break Rate

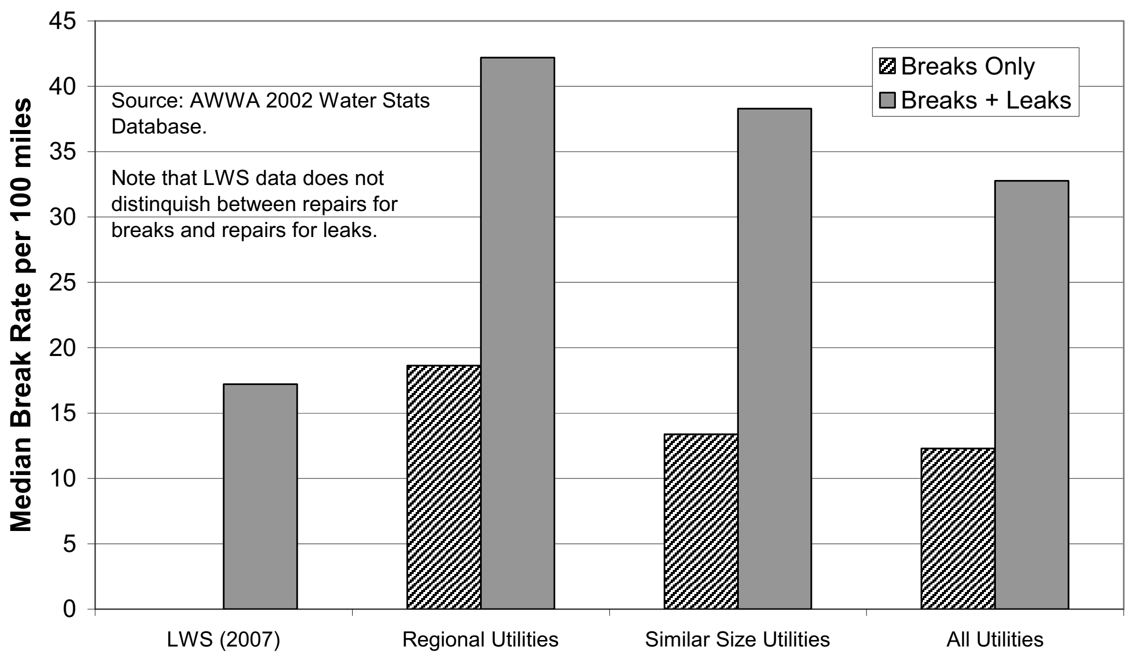
The LWS main break rate for 2007 was 17.2 main breaks per 100 miles, and the 10-year average break rate for 1998 to 2007 was 14.9. Historically, available data from LWS has not distinguished between main break and leak repairs. Therefore, the 2007 break rate is considered to include repairs for both breaks and leaks.

The Water:/stats database contains information about a variety of system characteristics that can be used to develop benchmarking groups. A total of three groups were selected for main break benchmarking comparisons with the LWS. A brief description of the groups and rationale for their selection follows:

- Regional Utilities includes eight reporting utilities in NE, MO, KS, and IA with a population of over 50,000.
- Similar Size Utilities includes 59 reporting utilities with distribution main length between 500 miles and 2,000 miles.
- All Utilities includes 222 reporting utilities after removing utilities with incorrect or no reported data, and with a reported length of at 100 miles or greater.

The Water:/stats database includes information on both the number of breaks and leak repairs per year. Although corresponding data for LWS is not yet available, comparisons between LWS and the selected benchmarking groups for both break repairs and break-plus-leak repairs is shown on Figure 7-5.

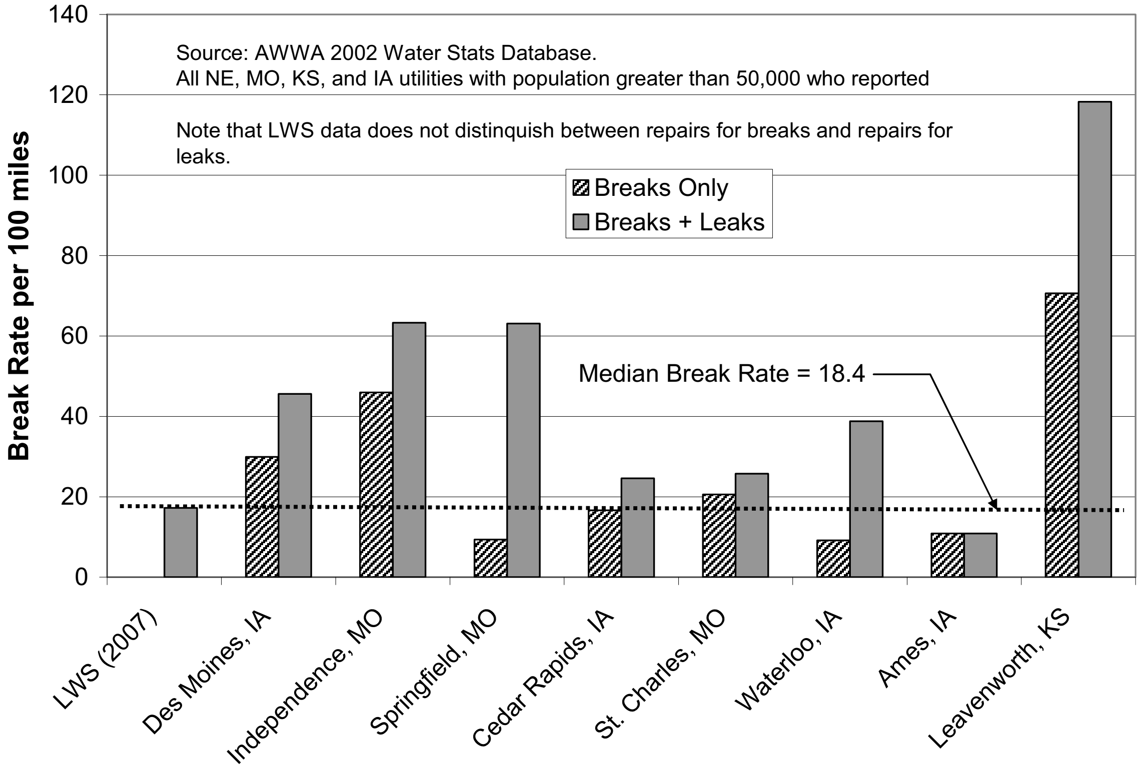
Figure 7-5 Benchmarking of LWS Break Rate



Regional Utilities includes eight utilities in NE, MO, KS, and IA with a population of over 50,000.
Similar Size Utilities includes 59 utilities with distribution main length between 500 miles and 2,000 miles.
All Utilities includes 223 reporting utilties after removing utilities with incorrect or no reported data.

An additional detailed comparison of the LWS break rate to other regional utilities in shown on Figure 7-6.

Figure 7-6 Comparison of LWS Break Rate to Regional Utilities



According to Figure 7-5, the LWS annual rate of repairs for leaks and breaks is slightly less that the median rate reported by other regional utilities, but slightly greater than that reported for the 59 utilities of similar size and all the reporting utilities. This comparison suggests that LWS main break rate is neither low nor excessively high.

The repair rates for main breaks and leaks should be viewed with caution as indicators of distribution system performance. They are best used with indicators of the relative performance of the system in terms of its effectiveness and efficiency in identifying/locating and repairing mains, as well as the overall system integrity in terms of water loss and level of service. For example, a relatively high repair rate may be consistent with utility efforts to achieve optimal balance between repair and replacement in order to minimize long-term capital and operation and maintenance costs.

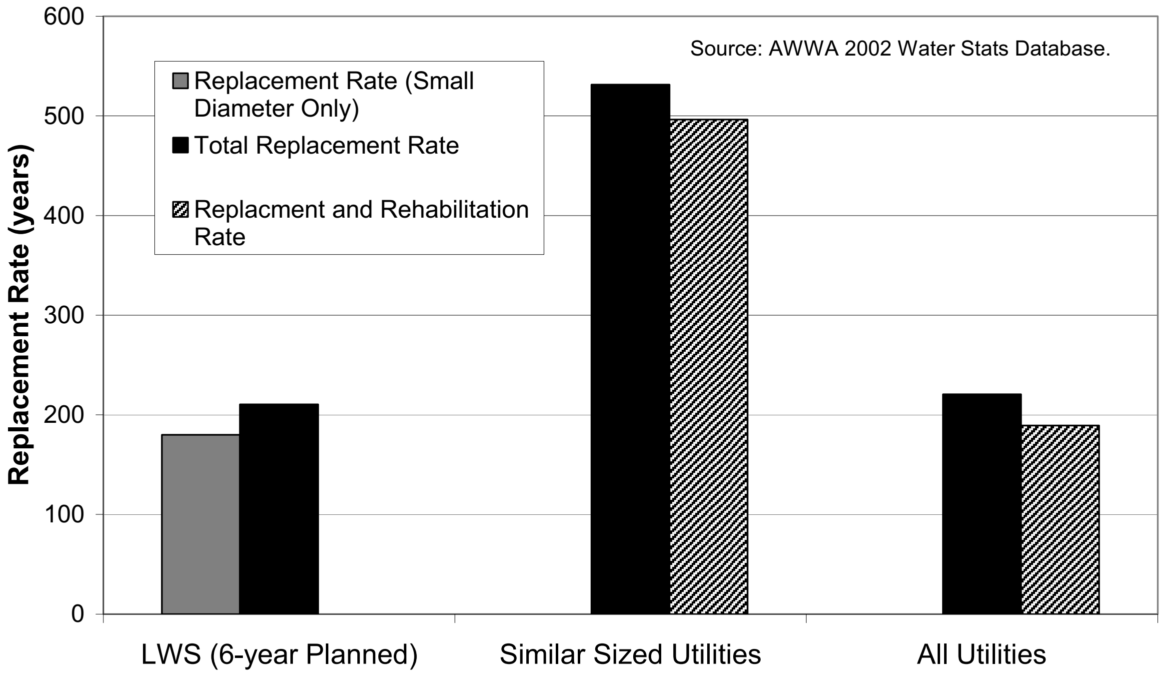
7.6.2.2 Replacement Rate

The rate at which a water utility is replacing its deteriorated distribution mains is a relatively common benchmark for water distribution systems. It is frequently expressed in terms of a replacement cycle – that is the length of time in years required to replace the total length of mains in a system. It can be calculated by dividing the total length of mains in the system by the replacement rate in miles per year.

The Water:/stats database includes information on the total number of miles of distribution mains that were replaced by each utility and the total length of main in the system. The database also includes the total length of main that had been rehabilitated via cleaning & lining, slip lining, pipe bursting, and cured in place piping (CIPP). This information was used to develop distribution main replacement rates. In addition, the effective renewal rate considering both main replacement and rehabilitation was calculated. Comparisons between LWS and the benchmarking groups are shown on the Figure 7-7. Benchmarking information for the eight regional utilities is not shown of Figure 7-7 because only four reported any replacement program, and of those four, the replacement rate was extremely high (1,600 years) to extremely low (less than 1 year), with the other two utilities reporting rates of 32 years and 341 years.

The Water:/stats database does not include information on small-diameter mains as a portion of the total distribution system. The replacement rate is therefore based on the total length of main in the system. The budgeted length of main replacement rate LWS for the next six years is about 5 miles per year. Based on replacement of the small-diameter finished water mains only, this corresponds to a replacement rate of 180 years.

Figure 7-7 Benchmarking of LWS Replacement Rate



LWS replacement rate calculated based on budgeted replacement rate of 5 miles per year for next 6 years.
LWS replacement rate is the planned replacement rate based on most current 6-year CIP.
Other utlities reported replacement rate only for utilities with a reported replacement programin 2002/2003.

The LWS replacement rate appears appropriate and is in line with the average rate for all utilities that report a replacement program.

A water utility's annual main replacement rate typically reflects a trade-off between the costs and risks associated with continued repair of main breaks and leaks with the upfront costs of main replacement. The appropriate balance between the two depends on a number of factors, including the relative cost of repairing vs. replacing mains.

7.6.2.3 Valve Inspection Program

Of the eight regional utilities, three report that all valves larger than 12-inch are exercised as part of an annual valve inspection program. Of these three, two report that all system valves are exercised annually, with the third reporting about 1/3rd of the small-diameter (12-inch and smaller) valves being exercised annually. Of the utilities of similar size, nearly 30 percent do not report an annual valve inspection program.

A total of 338 utilities reported the number of valves in their system. Of these, a total of 130 (nearly 40 percent) did not report an annual valve inspection program. The frequency of reported valve inspections, identified as the number of years to inspect valves was determined for this study and is shown on Figure 7-8.

Figure 7-8 Benchmarking of Valve Inspection Program

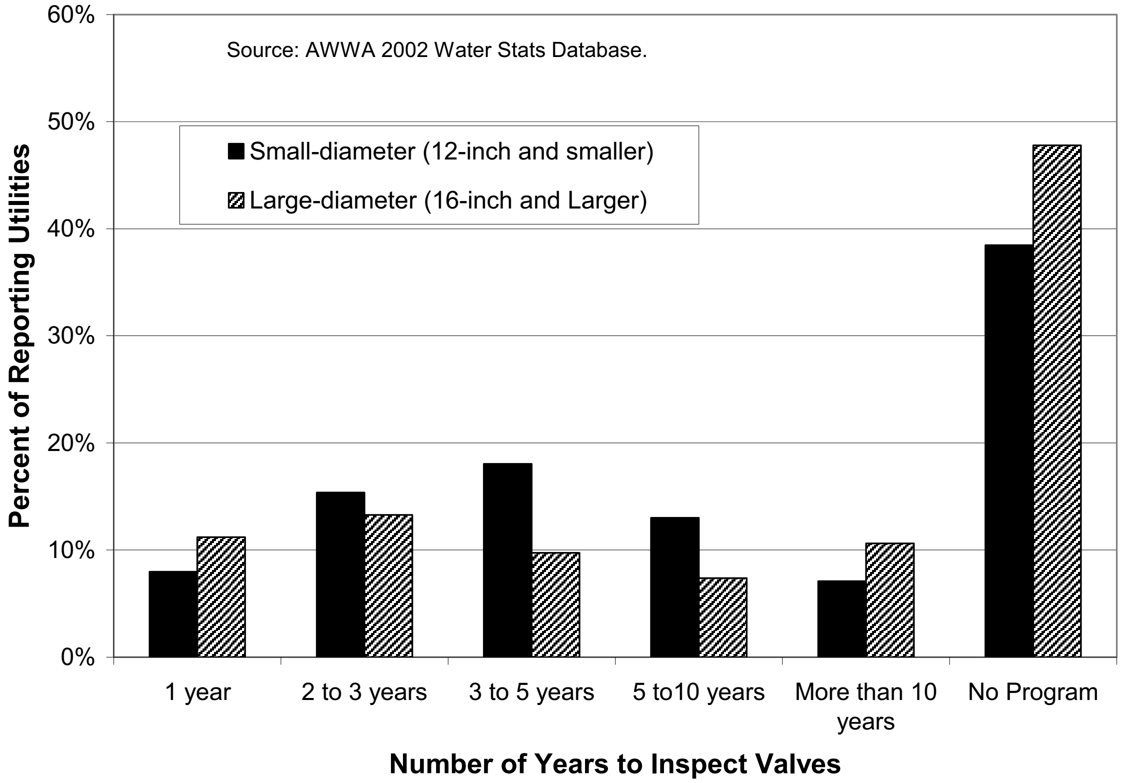


Figure 7-8 shows that the most common period of time to inspect small-diameter system valves is from three to five years, for utilities that report a valve inspection program. This is the period of time that Lincoln reports for their valve inspection program. Approximately 20 percent of LWS valves are inspected annually. However, large-diameter valves are generally exercised / inspected more frequently for those utilities that report a valve inspection program. Figure 7-8 shows that the most common period of time to inspect large-diameter valves is from two to three years. LWS reports that there is no special large-diameter, or critical, valve inspection schedule.

7.6.2.4 Hydrant Inspection Program

LWS attempts to inspect all fire hydrants on an annual basis. However, LWS reports that only about 80% were inspected in 2007. In the Water:/stats database, a total of 318 utilities reported the number of hydrants in their system. Of these 318, only 23 did not report a hydrant inspection/exercising program. The frequency of reported hydrant inspections, identified as the number of years to inspect hydrants was determined for this study and is shown on Figure 7-9.

Figure 7-9 Benchmarking of Hydrant Inspection Program

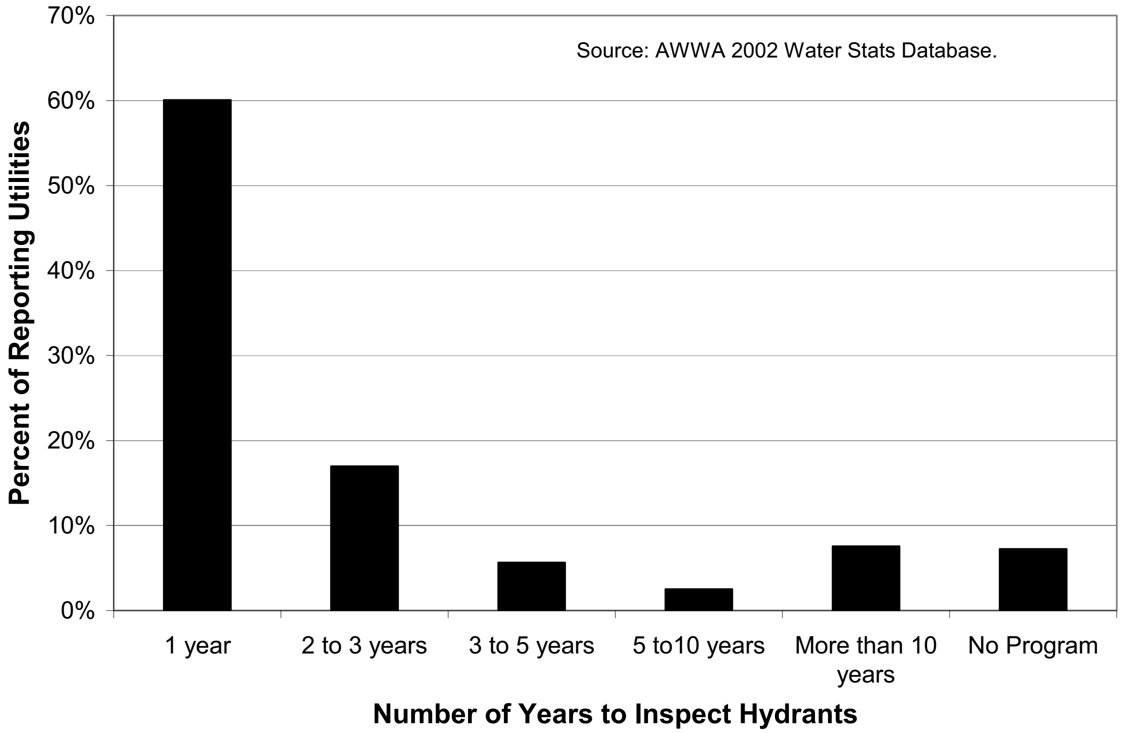


Figure 7-9 shows that the majority of utilities that report a hydrant inspection program attempt to inspect all hydrants on an annual basis, which is similar to that reported by LWS.

7.7 Assessment of LWS System Performance

The 1995 AWWARF report *Distribution System Performance Evaluation* suggests that distribution system performance measures are generally grouped into the three categories of adequacy, dependability, and efficiency. Adequacy refers to the delivery of an acceptable quantity and quality of water. Dependability measures the ability of the distribution system to consistently deliver an acceptable quantity and quality of water. Efficiency reflects how well resources such as water and energy are utilized. Measurement of performance is evaluated using performance measures. Performance measures are classified as structural, hydraulic, water quality, and customer perception. Several specific performance measures related to the issues presented in this report are discussed in the following sections.

7.7.1 Adequacy - Quantity and Quality

An adequate quantity of water at adequate pressure is being supplied to the customers. Some hydraulic bottlenecks have been identified in this study and improvements have been designed to rectify such deficiencies. Detailed fire flow analyses conducted for this report and described in Chapter 5 reveal that there are only limited areas that experience deficient fire flows – and these are being prioritized for improvements. The pressure at the customer tap varies from 35 psi to 110 psi. The LWS goal is to provide a minimum of 45 psi at the customer tap for new construction. There is also adequate fire flow capacity in the system. Internal corrosion and tuberculation, which is very prevalent in many major US cities and is a cause of water quality and hydraulic deficiency problems, is reported to be absent in the LWS distribution system.

The quality of water supplied to the customer meets and exceeds EPA requirements. While red water incidents occasionally occur in confined areas, it is not a serious problem. Infrequent positive Coliform test results have occurred in the past in the Belmont Service Level in the Air Park area. LWS has taken proactive measures to eliminate cast iron pipe in the area, and continues to monitor the area with an aggressive policy to eliminate water quality problems.

7.7.2 Dependability - Main Breaks and Main Replacement

While pockets of corrosive soils are abundant in the City service area, and especially near Salt Creek, steps have been taken to combat external corrosion. This includes polyethylene encasement wrapping of ductile iron mains, tape coating of steel mains in combination with cathodic protection, and use of PVC pipe. The LWS annual rate of repairs for leaks and breaks is slightly less that the median rate reported by other regional utilities, but slightly greater than that reported for the 59 utilities of similar size and all the reporting utilities. The 2007 main break rate of 17 main breaks per 100 miles is less than the reasonable goal stated in 1995 AWWARF report *Distribution System Performance Evaluation* of 25 to 30 main breaks per 100 miles per year.

LWS has a significantly expanded their small diameter main replacement program in the past decade, and is currently replacing about 0.55 percent of the small diameter mains each year. It would take approximately 180 years to replace the existing small diameter mains at the current rate. The current replacement rate is slightly greater than the general replacement needs for the US as outlined in the 2002 EPA *Clean Water and Drinking Water Infrastructure Gap Analy*sis report, and it significantly exceeds the median time of replacement as reported by 222 utilities in the 2002 Water:/stats database of over 500 years.

There are no formal criteria established for main replacement project. Most of the replacement candidates are selected based on their history and the judgment of staff. The main replacement program is also coordinated with the street paving program. In addition, the LWS staff meets twice a year with staff of major surrounding cities to exchange information about ongoing activities, concerns, and new initiatives. During such meetings, input is sought on factors considered in selecting mains for replacement.

7.7.3 Dependability - Valve Inspections

It is nearly impossible to quantify the reliability of a particular point in the distribution system, much less the reliability of the entire system. However, as described in the AWWA Manual M31 *Distribution System Requirements for Fire Protection,* a utility can minimize the effects of emergency shutdowns by confining the outage to the smallest possible area. To do this, the utility must have numerous valves, maintenance personnel who know the location of the valves and have ready access to records of their location, valve boxes that are free from debris, and valves that are in good operating condition. Only frequent exercising of valves can ensure this.

A review of other utilities valve inspection programs using the Water:/stats database shows that the LWS valve inspection program is generally consistent with other utilities in the US. However, LWS does not have a specific large-diameter, or critical, valve inspection program. Many utilities report that their large-diameter (16-inch and larger) are inspected more frequently than their small-diameter (12-inch and smaller) valves. Development and implementation of a large-diameter valve inspection program should be considered by LWS. Such a program may reduce the possibility of placing a large percentage of the customer base without water as a result of an inoperable large-diameter valve.

7.7.4 Dependability - Hydrant Inspections and Flushing

The LWS has a hydrant inspection program in place with the goal of flushing the system every year. However, in 2007 only about 80 percent of the hydrants were inspected. The Water:/stats database information reveals that a majority (60 percent) of US utilities attempt to inspect all their hydrants on an annual, or more frequent, basis. The frequency of hydrant inspections by LWS is generally considered good and is consistent with the practices of the majority of utilities in the US.

The AWWA Manual M17 *Installation, Field Testing, and Maintenance of Fire Hydrants* recommends that all fire hydrants should be inspected regularly, at least once a year, to ensure satisfactory operation. Manual M17 further provides recommended procedures for inspecting hydrants, and recommended record keeping for the hydrant inspections. Based on interviews with LWS staff, it does not appear that the inspections being conducted or the record keeping of hydrant inspections meet the requirements of the M31 Manual.

The Insurance Service Offices (ISO) reviews the fire suppression capabilities of a community and assigns a Public Protection Classification which impacts the insurance rates within a community. The overall rating is based on individual scores in three categories. The fire alarm and communications systems review comprises 10 percent of the total score. The Fire department review comprises 50 percent of the total score. And the water supply system comprises 40 percent of the score. The water supply system score is further determined by the ability to meet needed fire flows at each location (35 percent), the presence of a pumper outlet on each hydrant (2 percent), and the hydrant inspection frequency (3 percent). For maximum credit, all hydrants must be inspected twice a year, and records must be kept of the inspections. While increasing the inspection frequency of hydrants may improve the ISO classification, it would only have a small impact. More benefit may be obtained in the ISO rating by ensuring adequate operation of all hydrants, and implementing improvements to increase available fire flow in deficient areas.

The 1995 AWWARF report *Implementation and Optimization of Distribution Flushing Programs* indicates that of 281 utilities responding to a survey, more than half have flushing programs. Much emphasis was placed on customer complaints to identify and locate problems in the distribution system. This is consistent with the flushing program currently conducted by LWS. In addition to flushing hydrants which show high levels of color during the routine hydrant inspection program, other hydrants are flushed by LWS staff as needed in response to customer complaints. However, the practices of most US utilities including LWS are not necessarily consistent with the 1995 AWWARF report. The AWWARF report recommends that the entire distribution system should be thoroughly flushed at least once per year. Furthermore, it recommends that at all dead ends should be equipped with blow-off valves, and that at a minimum, blow-off valves should be flushed once per year at the beginning of the peak water use season. The current flushing program practiced by LWS does not thoroughly flush the system since hydrants are opened for only a very short amount of time, and there is no systematic approach to flushing the hydrants to achieve the most benefit.

7.7.5 Recordkeeping

A majority of the LWS historical records of existing buried infrastructure have been converted to an electronic format, but there are areas of the system that have insufficient data or the data is simply missing. Additionally, the length of main installed per year is not available prior to 1976. This makes it difficult to calculate the average age of the system. Lack of such data hinders the potential use of advanced predictive tools such as the AWWARF "KANEW" model and the "Nessie Curve" model for asset replacement funding. The LWS is currently engaged in an effort to populate information in the water system geodatabase to include the most accurate information on pipe diameter, material, and installation year. As of the writing of this report, this effort was reported to be about 50 percent complete.

A review of the geodatabase of main breaks recently prepared by LWS reveals that there are a number of duplicate records, and miscellaneous amounts of detail associated with main break records in the database. Detailed information is not collected from each main break repair. Only a limited amount of data is entered into the CMMS for each main break repair.

As indicated above, it does not appear that the record keeping of hydrant inspections meet the requirements of the AWWA Manual M17.

7.8 Recommendations

7.8.1 Replacement Funding

Preliminary investigations conducted for this report, and experience with other utilities indicates that the current main replacement program is likely adequate for the near future. However, significant increases may be required in the future.

Additional investigations should be conducted to better quantify the level of replacement funding required in the future. These investigations should evaluate the life expectancy of various pipe materials in the system, quantify the amount of pipe by age and material, establish level-of service criteria (customer outages, water quality), evaluate the cost effectiveness of repairs versus replacement, and conduct a cost-effective analysis to determine optimal replacement timing for the various materials under various conditions.

Consideration should be given to developing a comprehensive "asset management plan" to establish future fiscal needs for preservation of LWS assets. Asset management allows minimizing the total cost of owning and operating the system while maintaining the quality of the service the customer desires.

7.8.2 Small-diameter Rehabilitation and Replacement Planning

Concurrent with additional investigations to establish the level of replacement funding, LWS should consider conducting a detailed pipeline replacement plan. The plan should establish formal replacement criteria and prioritize main replacement candidates based on several criteria including physical, functional, and impact categories. These investigations should establish clear trends correlating incidences of both physical and functional deterioration of mains to underlying pipe characteristics and/or environmental factors. The amount of annual main replacement projects would be based on the recommended funding levels.

Consideration should be given to replacement by trenchless techniques as well as rehabilitation of existing lines. These techniques may include pipe bursting, sliplining, modified sliplining, and cured-in-place lining in place of replacement by open cut methods. These techniques have shown the potential of producing significant savings in some locations. Such savings could be used to replace or rehabilitate a larger portion of the distribution system annually.

7.8.3 Large-diameter Inspection Planning

Concurrent with the investigations to evaluate replacement funding and prioritize mains for replacement, LWS should evaluate and formulate a large-diameter inspection program. When large diameter pipes reach the point of failure they have the potential for significant damage to the surrounding area simply due to the volume of water present. Several inspection techniques exist to proactively predict or prevent these failures. The following inspection techniques should be considered as part of a formal large-diameter inspection program.

- Acoustics monitoring.
- Sahara® leak detection.

- Ultrasonic Thickness Measurements.
- Broadband Electromagnetic Probe (BEM).
- Closed Circuit Television (CCTV) inspection.
- External deterioration, if suspected, can best be assessed by potential surveys.
- Soil Testing.

7.8.4 Maintenance Activities

As the system continues to expand and resources are held constant, some critical maintenance and repair procedures may be not be performed or may be delayed. If sufficient resources are not available, operability of the system may be impacted. The existing maintenance activities should be given high priority to continue, and the following maintenance and inspection activities should be considered for incorporation into the routine maintenance program for distribution system facilities.

- Modify the valve inspection schedule to inspect large-diameter valves (16-inch and larger) once per year
- Modify hydrant inspection procedures and record keeping to be consistent with the recommendations of AWWA Manual M17 *Installation, Field Testing, and Maintenance of Fire Hydrants*.
- Implement a flushing program for dead-end mains and the entire system in accordance with the recommendations of the 1995 AWWARF report *Implementation and Optimization Of Distribution Flushing Programs.* Implementation of a formal flushing program should consider uni-directional flushing to maximize effectiveness.
- Implement a leak detection and monitoring program to identify system leaks and reduce non-revenue water.
- Implement a corrosion protection monitoring program.

7.8.5 Recordkeeping

A standardized form should be developed and required for collection of data during all main break repairs. The form should be based on that included in the *2002 Facilities Master Plan* and at a minimum should include the following fields:

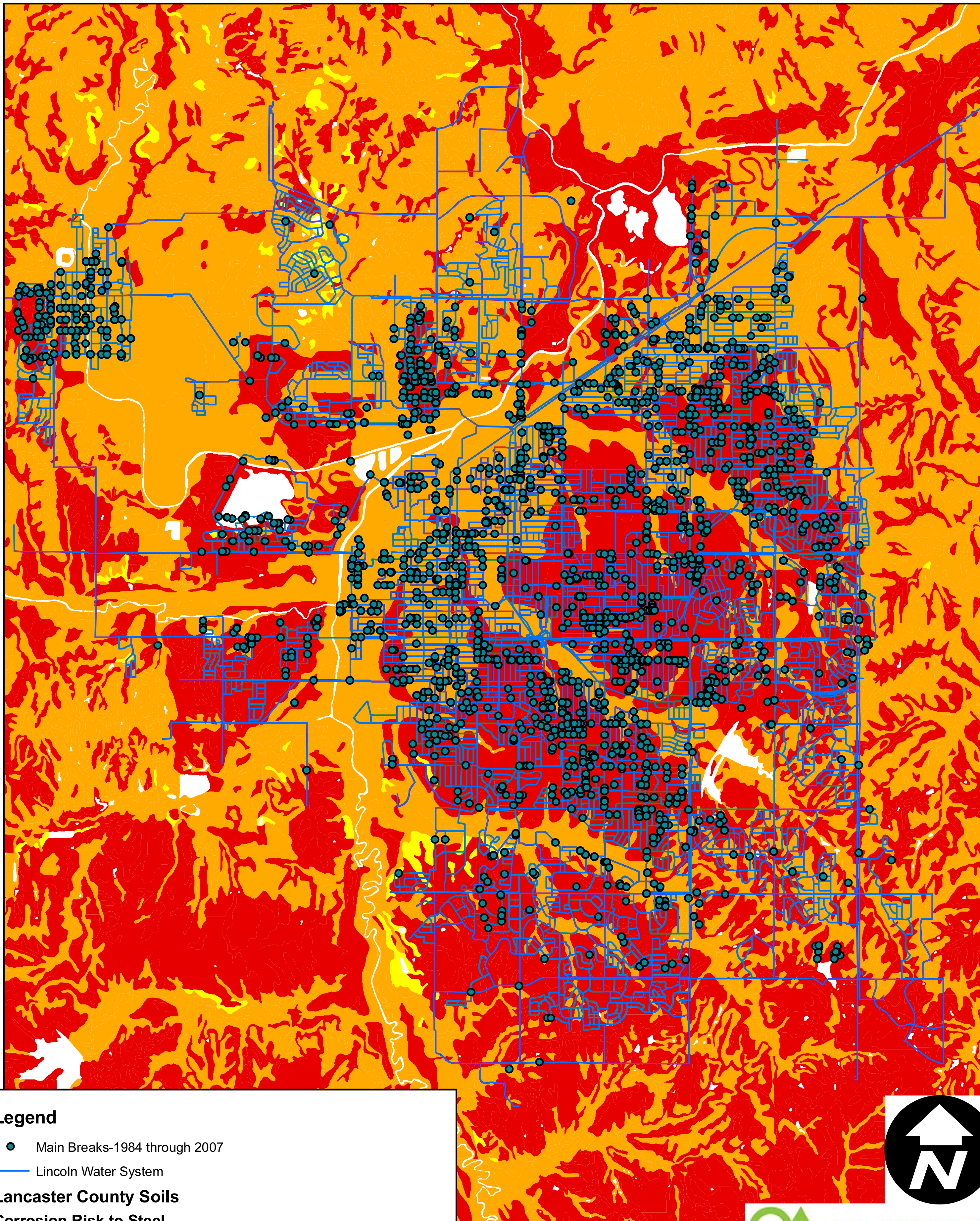
- Location of break (street address, map number, etc.).

- Date and time of break report and break repair.
- Type of break (joint failure, pipe failure, etc.).
- Type and size of pipe (cast iron, ductile iron, pvc, etc.).
- Depth of cover.
- Year of pipe installation.
- Bedding type (soil, aggregate, rock, etc.).
- Soil condition.
- Corrosion protection, if any
- Possible cause of break.
- Surrounding land use (for an indication of traffic loads on the pipe).
- Repairs made (leak clamp, welded, recaulked joint, replaced pipe section, etc.).

Standardized forms should be developed and required for collection of data during all hydrant and valve inspections. The forms should be based on those provided in the AWWA Manual M17 *Installation, Field Testing, and Maintenance of Fire Hydrants*.

All information collected during main break repairs and valve and hydrant inspections should be entered into the CMMS. Consideration should be given to providing field crews with laptop computers to collect the data to facilitate ease of entry into the CMMS.

The existing records in the main break geodatabase should be cleaned up to remove duplicate records. All the information contained in the field forms for historic main breaks that have not been entered into the geodatabase should be entered. This procedure should continue up to the time that the information is entered into the CMMS as recommended above. The information from the database can then be used in the development of a main rehabilitation and replacement program to assess breaks by material, location and other considerations.



Legend

- Main Breaks-1984 through 2007
- Lincoln Water System

Lancaster County Soils

Corrosion Risk to Steel

- Low (Pipe Wall Thickness Loss of 2-15 Micro-Meters per Year)
- Moderate (Pipe Wall Thickness Loss of 15-50 Micro-Meters per Year)
- High (Pipe Wall Thickness Loss of 50-200 Micro-Meters per Year)

OLSSON ASSOCIATES

Soil Corrosion Risk Map

Figure 7-3

8.0 Recommended Improvements

8.1 General

Based on the findings of the steady state hydraulic analyses, the water age analyses, the fire flow analyses, and the main replacement program review; a comprehensive capital improvements program was prepared. This comprehensive CIP includes budget costs and is staged and prioritized to identify reinvestment needs and improvements for additional capacity and reliability through year 2057. Recommended improvements to address rehabilitation/replacement projects are prioritized and listed separately.

It should be recognized that the alignments shown for the recommended improvement mains are approximate locations. Specific street locations for the mains should be determined during the preliminary design. 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.

8.2 Cost Estimates

In every 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.

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 various components of project costs are considered in the following paragraphs.

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 acquisition is a significant activity that determines whether a project occurs, the cost of land acquisition 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 20 percent of the construction cost has been included.

Engineering services may include preliminary investigations and reports, site and route surveys, 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 20 percent allowance for engineering services, legal, and administrative costs.

8.2.1 Basis of Costs

In considering the estimates presented in this report, it is important to realize that they are reported in year 2007 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 7956, which is the annual average value for year 2007 (though November). 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 7956.

8.2.2 Pipelines

The *2002 Facilities Master Plan* used a construction cost of $5.00 per diameter-inch per lineal foot plus a 20 percent contingency for the basis of pipeline construction costs. A review of the ENR average annual CCI shows that the CCI has increased from 6694 in 2003 to 7956 in 2007. This represents an increase of 19% over that 4-year time period. A large diameter water transmission main project was recently bid, in the later part of 2007, and the low bid came in at a unit cost of $6.47 per diameter-inch (excluding installation of a fiber-optic cable and construction of a pressure regulating station). This large diameter main will be constructed in a generally rural, undeveloped corridor.

For this 2007 update, opinions of probable construction costs for main improvements in currently undeveloped areas are based upon a unit costs of $6.25 per diameter-inch per lineal foot plus a 20 percent contingency (6.25 x 1.2 = $7.50 per diameter-inch when including contingencies). Probable project costs are calculated by adding a value equal to 20 percent of the total construction cost (including contingencies) for engineering, legal and administrative costs. The total value for probable project costs in currently undeveloped areas is therefore $9.00 per diameter-inch per lineal foot.

As indicated in the 2002 Master Plan, 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, for residential service. As a general guideline, the cost of one mile of 16-inch main would be about $760,000 and the cost of one mile of 24-inch main would be about $1,140,000. Interior gridding of 12-inch mains within the one-mile grid are identified in this report for the Phase II (Year 2019) development area, but this report includes costing for only 16-inch pipes and greater because developers will pay most of the cost associated with construction of 12-inch mains in new development areas.

For construction in fully developed and congested areas, a project cost of $14.00 per diameter-inch per lineal foot was used except for improvements relating to fire flow deficiencies which costs were defined on an individual basis depending on location and diameter. These unit costs and individual fire flow costs typical constitute an allowance for street removal and replacement as well as additional coordination with other utilities.

8.2.3 Pumping

The total opinion of construction costs for a booster pumping station are based on a net cost of $190,000 per mgd of installed capacity plus 20 percent for engineering, legal, and administrative costs. This is based on typical Lincoln Water System pumping stations with permanent structure and sized for expansion. Therefore the total probable project cost for new pumping stations is $230,000 per mgd.

The construction costs of installing a new pump in a pumping station which is designed for the addition of a pump, or for replaced a pump in an existing pumping station, are based on a unit cost of $40,000 per mgd of installed capacity. This cost includes the addition or replacement of electrical equipment. Probable project costs are calculated by adding a value equal to 20 percent of the total construction cost for engineering, legal and administrative costs. Therefore the total probable project costs for capacity increases at existing pumping stations is about $50,000 per mgd.

8.2.4 Storage

The project cost for distribution system storage varies considerably, depending on such factors as type, material, capacity and support system. Estimated total unit project costs were developed for three types of facilities that are similar to those currently in service. The estimated total unit project costs include site work, reservoir foundation, the reservoir, site piping, controls and miscellaneous appurtenances.

Steel or pre-stressed concrete ground level reservoirs would be used primarily for larger reservoirs having capacities of over 2 MG and may be above-grade or buried below-grade. The construction cost of an above-ground ground level reservoir is based on a unit construction cost of $0.70 per gallon plus a 20 percent contingency. Probable project costs are calculated by adding a value equal to 20 percent of the total construction cost for engineering, legal and administrative costs. Therefore the total probable project cost for an above-grade ground level reservoir is about $1.00 per gallon.

The construction cost of a buried below-grade reservoir is based on a unit construction cost of $1.05 per gallon plus a 20 percent contingency. Probable project costs are calculated by adding a value equal to 20 percent of the total construction cost for engineering, legal and administrative costs. Therefore the total probable project cost for a buried below-grade ground level reservoir is about $1.50 per gallon.

The construction cost of elevated reservoirs is based on a unit cost of $1.40 per gallon plus a 20 percent contingency. Again, probable project costs are calculated by adding a value equal to 20 percent of the total construction cost for engineering, legal and administrative costs. Therefore the total probable project cost for elevated reservoirs is $2.00 per gallon.

8.2.5 Pressure Reducing Valve Stations

Pressure reducing valve (PRV) stations transfer water from a higher service level to the next lower service level. It is assumed that the piping, valves, electrical and instrumentation components (including a flow meter) for a PRV station will be housed in a below-grade concrete vault structure. The total project cost for each PRV station is estimated to be $75,000.

8.2.6 Pressure Monitoring Stations

Pressure monitoring stations are used monitor pressures in the distribution system in areas of interest. It is assumed that the electrical and instrumentation components will be housed in small pre-packaged structure including a small enclosure located above grade, complete with necessary instrumentation. The total project cost for each pressure monitoring station is estimated to be $20,000.

8.3 Long-range Plan (Year 2057)

A long-range plan was developed based on providing water service to the year 2057 service area. The year 2057 service area includes development into the Tier II development limits as identified in the recent *2030 Comprehensive Plan* prepared by the Lincoln and Lancaster County Planning Department. Future service level boundaries were determined for the Tier II limits and projected water demands were developed which were used to determine needed pipelines and facilities. Hydraulic analyses using the 2007 Model were used in conjunction with previous evaluations as conducted for the *2002 Facilities Master Plan* to develop a long-range plan for design years 2019 and 2032 were then coordinated with the long-range plan.

8.3.1 Long-range Plan Supply

The projected year 2057 maximum day demand of 224 mgd exceeds the ultimate planned capacity of the existing water treatment plant supply and treatment facilities, and the transmission capacity between the water treatment plant and Lincoln, all of which are expected to maximize at 210 mgd.

Additional supply beyond 210 mgd is being pursued by LWS, and is expected to be delivered to the distribution system somewhere in the southeast portion of the city. This future supply was considered in the long-range plan and the other phased improvements developed for this report. However, the long-range plan developed for the report did not evaluate specific facilities to deliver the future water supply to the distribution system. Future master planning efforts should consider the timing and sizing of the facilities required to deliver a second source of supply to the City at a location in the southeast portion of the City.

8.3.2 Long-range Plan Service Levels

Much of the area within the Tier II limits can be provided adequate pressures from the existing service levels. However, some areas will require the establishment of new service levels.

High ground areas on the northern and western side of the year 2057 service limits cannot be provided adequate pressures from the existing Belmont Service Level and will require booster pumping. To provide adequate pressure to these high ground areas, the existing Northwest Service Level is recommended to be expanded to serve in the northwest areas, and a future Southwest Service Level is recommended for the southwest areas.

There are several small areas of high ground elevation on the eastern side of the year 2057 service limits that cannot be provide adequate pressures from the High Service Level and would require booster pumping. Future pumping and storage facilities are not identified in this report due to the limited size of these areas. Consideration should be given to restricting development in these areas.

8.3.3 Long-range Plan Pipelines

A significant issue in the development of the long-range plan was the need to provide service to a large potential growth area south of the existing Belmont Service Level and west of Wilderness Park. The existing distribution system does not have adequate capacity to provide for any significant growth in demands in this area. A new transmission main, hereafter referred to as the northwest loop transmission main, should be constructed around the northwest portion of the existing service area to supply the potential increased demands on the southwest side of the system. The northwest loop transmission main will require a major investment and was developed for this study with sufficient capacity to provide for expansion to other growth areas and to serve fuller development in the southwest area beyond 2057. Supply to the northwest loop transmission main would be provided by a pumping station located at the existing Northeast Reservoir and Pumping Station.

Service to significant growth areas on the east and south portions of the 2057 Tier II service limits must consider the planned expressway along the east and south sides of the city. Sufficient highway crossings of significant capacity should be provided under the planned expressway to provide adequate room for future growth

8.3.4 Long-range Plan Storage Reservoirs

Recommended storage improvements for the long-range plan are described below.

- High Service Level. The High Service Level has only 9 MG of existing floating storage facilities resulting in the lowest ratio of available floating storage to average day demand of any service level. It would be the first service level to run out of water upon complete loss of power to the City. A total recommended 10 MG of storage should be provided at two locations. The first additional 4 MG of storage is recommended to be constructed by year 2032 in the southern portion of the High Service Level to meet demands and maintain adequate service levels. This additional storage volume will also provide for improved reliability under emergency conditions. A second 6 MG of storage is recommended to be constructed after 2032 in the northeast portion of the High Service Level.

- Northwest Booster District. In order to discontinue the operation of the Northwest Booster District as a closed system, it is recommended that floating storage be provided before the year 2032. An elevated reservoir with a volume of about 1 MG and an overflow elevation of 1460 feet should be provided in the Northwest Booster District. High ground of 1320 feet elevation is located just east of NW 12th Street north of existing development. Upon Tier II development on west side the Belmont Service Level, the process of constructing connective piping between these two areas should commence in order to convert the entire higher ground areas around the northwest sides of the existing Belmont Service Level to one service level. An additional 1 MG elevated storage reservoir should be provided for this expanded Northwest Booster District after 2032.

- Southwest Service Level. High ground in southwest portion of the year 2057 service area will require booster pumping from the Belmont Service Level to maintain adequate pressures. A future Southwest Service Level should be created to serve this area, the it should be provided with an elevated reservoir. At year 2057, the aggregate development density of this southwest area is low, and only about 1 MG of storage is required to meet year 2057 demands. However, there is significant area that can continue to develop and based on the aerial extent about 2.0 MG of storage should be provided for ultimate development. The elevated reservoir should have an overflow elevation of 1500 feet. The area within this future Southwest Service Level is not expected to develop until after 2032 and the location and size of this storage volume should be reassessed in future master plans.

- Belmont Service Level. Additional storage should be provided to serve the southwest portion of the Belmont Service Level. The reservoir should have an overflow elevation of 1400. However, during high demand periods, it will be difficult to maintain the level in the reservoir while adequately fluctuating the existing Air Park and Northwest Reservoirs. Under these conditions the water level in the reservoir would be allowed to drop about 5 feet to 15 feet below the levels in the other Belmont Service Level reservoirs. Based on this operation, a 5.0 MG ground level reservoir with a sidewater depth of 60 feet should be provided. Ground elevations no higher than 1270 feet elevation should be provided service by the reservoir. Higher ground elevations should be provided service by the booster districts discussed above.

8.3.5 Long-range Plan Pumping Facilities

Several existing pumping stations will require increases in capacity, while several new pumping stations will be required to meet year 2057 demands. Several of these expansions and new facilities will be required to be constructed by 2032. Recommendations for expanded or new pumping facilities are provided below:

- Northeast Pumping Station. The long-range plan will require significant additional pumping from the Northeast Reservoir. A new pumping station in separate structure should be constructed to allow for pumping directly to both the Belmont and High Service Levels. Upon completion of this pumping station and the northern portion of the northwest loop transmission main, the existing Merrill Pumping Station should be retired. Pumping to the Belmont and High service levels will require pumps with rated total dynamic head of around 300 feet to 350 feet. A minimum firm capacity of about 38 mgd should be provided to the

Belmont Service Level. A minimum firm capacity of about 20 mgd should be provided to the High Service Level. This new pumping station would not be required until after year 2032.

The future pumping station at the Northeast site should also contain space for a minimum of 14 mgd of additional transfer pumping to the 51st Reservoir. The 36-inch transmission main from the water treatment plant to the 51st Reservoir is the oldest of the transmission mains from the water treatment plant into Lincoln. When this main reaches the end of its useful life and is abandoned, a replacement main should be constructed from the Northeast Reservoir to the 51st Reservoir and additional transfer pumping should be provided at the Northeast Reservoir. As stated earlier in this chapter, hydraulic analyses verify that the planned ultimate water treatment plant capacity of 210 mgd can be delivered to the Northeast Reservoir through the long range planned transmission lines. Future evaluations should be conducted to evaluate the capacity and head requirements for additional pumping capacity, considering other recommended and potential pumping unit modifications at the South Pumping Station.

- Vine Pumping to Southeast Service Level. A third pumping unit should be installed to pump from the "new" Vine Pumping Station to the Southeast Service Level. The third unit should be rated at 20 mgd at 210 feet. In addition, one of the 10 mgd units should be replaced with a 20 mgd unit, to provide a firm capacity of 30 mgd. The initial installation of a new 20 mgd unit is recommended to be installed as a late Phase II improvement, around 2019. Replacement of one of the 10 mgd units is recommended as a Phase III improvement.

- "A" Pumping Facilities. As described in the *2002 Facilities Master Plan*, about 31 mgd of water can be transferred from the 51st Pumping Station to the "A" Reservoirs by operation of two of the three 15 mgd units. An additional 15 mgd can be delivered from Vine to the "A" Reservoirs (without pumping at Vine) resulting in a total delivery to "A" of about 46 mgd. The required firm rated capacity to the High Service Level for this long-range plan is about 25 mgd which requires the existing firm pumping capacity to the High Service Level at "A". In order to provide sufficient capacity for the long-range plan, a new satellite pumping station should be constructed to pump to the Low Service Level. The new satellite pumping station should be rated at about 10 mgd with a total dynamic head of 155 feet.

- Pioneers Pumping Station. The Pioneers Pumping Station has been constructed with space for four pumps. Only three units have been installed; one at 2 mgd, one at 3 mgd, and one at 5 mgd; to provide a firm station capacity of 5 mgd. Consistent with the *2002 Facilities Master Plan*, a fourth unit rated 5 mgd is recommended to provide a firm capacity of 10 mgd. This fourth unit is not required until after 2019 and is a recommended Phase III improvement.

- Yankee Hill Pumping Station. The existing Cheney Booster Pumping Station is a buried station and was constructed with the intention that it would be temporary and eventually be replaced by a permanent above ground structure at the location of the Yankee Hill Reservoir. The future Yankee Hill Pumping Station should contain space for four pumping units, with two units rated 2 mgd and two units rated 4 mgd to provide a firm pumping capacity of 8 mgd. The units should have a rated head of about 110 feet of head. The existing Cheney pumping station is still providing adequate service, and has sufficient capacity to meet projected year 2032 maximum day demands. However, for purposes of this Master Plan, the future Yankee Hill Pumping Station is recommended as a Phase III improvement to be constructed before year 2032. At that time, the Cheney Booster Pumping Station could remain in place as a backup pumping station. If excessive maintenance becomes apparent after year 2032, or if safety issues interfere with providing adequate maintenance for the buried Cheney Pumping Station it could be removed.

- Booster Pumping Stations from Belmont. A future booster pumping station is identified in the long-rang plan for the expanded Northwest Service Level, and another booster pumping station is identified to provide service to the future Southwest Service Level. The areas which will require the new booster pumping stations are all located beyond the Tier I development limits and therefore no new facilities will be required until after 2032. The required capacities of these stations should be evaluated in greater detail in future master planning efforts.

- WTP Additional Pumping. Additional WTP High Service Pumping will be required as growth occurs. A new Pump No. 13, with a rated capacity of 20 mgd and rated head of 350 feet (similar to the existing Pump No. 11 and Pump No. 12) should be installed by year 2019. The addition of Pump No. 13 will fill all existing high service pumping bays WTP. Two more additional pumps rated at 20 mgd and 350 for head should be installed by the year 2032. One of these pumps should replace the existing Pump No. 10 which has lower discharge head characteristics than the other high service pumps. The addition of a new Pump No. 14 will require the construction of a new pumping station at the WTP. This station has been planned to deliver to the distribution system the ultimate planned capacity of the WTP of 210 mgd. The new pumping station should contain space for three pumping units sized for 20 mgd each.

8.4 Recommended Phased Improvements

Following development of the long-rang plan, a series of analyses were conducted to develop a recommended phased improvements to resolve current deficiencies, to meet projected demands, and to improve water quality. The phases of the program are summarized below.

- The "Phase I – Immediate Improvements" are those that have been identified as higher priority as a result of their immediate need or as a result of currently anticipated development. Phase I improvements also include improvements to correct identified fire flow deficiencies.
- Improvements recommended to meet year 2019 demand conditions are referred to as "Phase II – 12-year Short-term Improvements". The Phase II improvements will extend service to the limits of the Tier I –Priority B area.
- Improvements recommended to meet year 2032 demand conditions are referred to as "Phase III – 25-year Mid-term Improvements". The Phase III improvements will extend service to the limits of the Tier I – Priority C area.
- Improvements recommended to provide service beyond the Tier I limits out to the Tier II limits are referred to as "Phase IV – 50-year Long-term Improvements".

A detailed tabular summary of information for the 16-inch and larger recommended improvement mains for Phase I, II, and III; and for the larger recommended improvement mains for Phase IV.

8.4.1 Phase I and Phase II Improvements (by Year 2019)

Phase I and Phase II recommended improvements will provide service to the limits of Tier I – Priority A and B development areas.

The Phase I immediate improvements should be viewed as a subset of the Phase II 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 capital improvement program (CIP). Some Phase II improvements that are not specifically identified as Phase I will also be included in the 6-year CIP and should be prioritized based on known or anticipated development.

8.4.1.1 Phase I – Immediate Improvements

The LWS current 6-year CIP was reviewed and compared to the results the base-year analyses and fire flow analyses which identified existing deficiencies, and to the year 2019 analyses which identified Phase II improvements. Based on this review, Phase I improvements were identified. The Phase I improvements should be included in the 6-year CIP regardless of specific development issues, and include the following:

- 36-inch main in Yankee Hill Road. Base year analyses identified this main as one of the highest priority improvements. It is required to maintain pressures in the southwestern portion of the distribution system under peak demand conditions.
- Improvements on NW 56th Street. LWS currently planned improvements include an improvement on NW 56th from O Street to Partridge Lane. It is recommended that this improvement should extend a couple additional blocks north to Aurora Street. In addition, roadway improvements are planned on NW 56th Street south of Adams Street; and main improvements are planned to be installed concurrent with this roadway improvement.

  LWS currently planned improvements include a main in the vicinity of SW 27th Street between A Street and O Street to provide a reliability loop for the customers in the Belmont Service Level. As an alternative to this improvement, a main should be constructed on SW 56th Street between A Street and O Street. This improvement will further help to provide the beginning of the long-term northwest loop transmission main project. In conjunction with the suggested on W 56th Street, a 16-inch main on A Street from SW 56th Street to SW 40th Street is recommended to complete the reliability looping.

- Northern Development Area. Improvement mains are required to provide service to the development area located north of I-80 between 40th and 56th Streets. Service to this area was evaluated in the March 2006 Development Study completed by Black & Veatch. This area should be provided service at pressures equivalent to the Belmont Service Level. A booster pumping station should be constructed with the first development to take place on the site, or, if the ground elevations of individual buildings allow for acceptable pressures under normal conditions, automatic sprinkler fire protection provisions should be provided for each individual building. If a booster pumping station is required, and in accordance with the March 2006 Development Study, the station should have a firm capacity of 3.6 mgd and should be constructed west of 56th Street at Interstate 80.

- Control Valve at Pioneers Pumping Station. This valve should be considered to provide for better control of the Pioneers 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, and manual override of the valve should be available at the WTP control center.=

- Fire Flow Improvements. Only six specific recommended improvements are identified to correct potential fire flow deficiencies in five areas. Actual fire flow goals should be verified for other potentially deficient areas before implementing improvements. Fire flow improvement project costs including a 20 percent contingency and 20 percent allowance for engineering, legal, and administrative costs are based on a unit project cost of $120 per foot in developed areas or very short lengths, and $80 per foot in undeveloped areas. The recommended fire flow improvements are summarized in Table 8-1.

Table 8-1 Recommended Improvements for Fire Flow Deficiencies				
ID	**Description**	**Unit Cost ($/ft)**	**Length (ft)**	**Project Cost**
F1	12-inch on Partridge between NW 58th and NW 57th	$110	590	$65,000
F2	Complete 8-inch connection on Aurora at NW 56th	$120	100	$12,000
F3	Replace 4-inch on D between S.18th and S. 20th with 6-inch	$120	1000	$120,000
F4	6-inch on N 53rd, north of Huntington (May not be required if building is sprinklered)	$120	600	$72,000
F5	6-inch looped main at SL boundary, N 53rd between Leighton and Huntington	$120	700	$84,000
F6	8-inch looped main at SL boundary, Fletcher between NW 15th and NW 12th	$120	900	$108,000
	Total Fire Flow Improvements			**$461,000**

(1) All project costs are reported in year 2007 dollars, and include an 20 percent contingency, plus a 20 percent allowance for engineering, legal, and administrative costs.

Opinions of probable project cost for Phase I main, storage and pumping improvements are summarized in Table 8-2 rounded to the nearest $10,000.

Table 8-2 Phase I Recommended Improvements			
Description		**Units**	**Project Cost**
Water main on Yankee Hill Rd		10,800 ft	$3,500,000
Water main on W 56th, A St. to Adams St.		16,200 ft	$5,250,000
Water main on Adams, NW 56th St to NW 48th St		1,200 ft	$170,000
Water main on A St., SW 56th St to SW 40th St		5,200 ft	$750,000
Water main west of N 56th, Arbor Rd to Bluff Rd.		6,200 ft	$1,340,000
Subtotal Phase I Main Extensions			**$11,010,000**
Fire Flow Improvements (see table 8-1)		Various	$460,000
Booster Pumping Station at I-80 west of N 56th St		5.6 mgd	$1,290,000
Control Valve in Pioneers Pumping Station		1	$50,000
Total Phase I Improvements			**$12,810,000**

(1) Phase I identified improvements are recommended to be constructed within the next 6-years and should be included in the LWS 6-year CIP. However, these Phase I improvements account for only a portion of the total main improvements included in the 6-year CIP. Other distribution system mains are required to support growth and should be prioritized by LWS based on locations of known developments.

(2) All project costs are reported in year 2007 dollars, and include an 20 percent contingency, plus a 20 percent allowance for engineering, legal, and administrative costs.

8.4.1.2 Phase II – Short-term Improvements

A significant amount of main extensions will be required by year 2019. About $61 million of total piping improvements are recommended as Phase I and Phase III (by year 2032). This equates to an average of about $5.0 million per year of required CIP funds for Phase I and Phase II pipeline extensions.

The most significant of the recommended Phase II mains is the main from the Northeast Pumping Station to Vine Reservoir. This main will increase transmission capacity to the distribution system and support growth on the east and south portions of the service area. This main was recommended in the *2002 Facilities Master Plan*, and about 1-mile of it has already been constructed. About five miles remain to be constructed and is recommended for construction around 2015 to 2016.

No storage improvements are recommended by year 2019. Pumping station improvements were previously described in the description of the long-range plan and are summarized below:

- <u>Vine Southeast Pumping Station.</u> A third pumping unit should be installed at the Vine Southeast Pumping Station. It should be similar to the existing two units, rated 20 mgd. This will increase the firm pumping capacity to the Southeast Service Level, from the Vine Pumping Station, to 20 mgd.
- <u>New "A" Satellite Pumping Station.</u> A new satellite pumping station should be constructed to deliver water from the "A" Reservoirs to the Low Service Level. The pump should be rated 10 mgd. This will increase the firm pumping capacity to the Low Service Level from the "A" location to 28 mgd. The station should be equipped with emergency standby power.
- <u>New WTP High Service Pump.</u> A new Pump No. 13 with a rated capacity of 20 mgd and head of 350 feet should be added at the water treatment plant. This pump will increase the firm capacity of high service pumping to about 107 mgd.

LWS currently has eleven pressure points monitored throughout the system separate from pressures monitored at distribution system pumping and storage facilities. As development occurs it is recommended that as a Phase II cost, several more of these monitoring stations should be included. Five future pressure monitoring stations are recommended with the locations of these to be determined. It is suggested that at least one of these stations should be placed in the Northwest Booster District and another should be located toward the southernmost portion of the Southeast Service Level. Most of the existing pressure monitors are used to monitor low pressures. Consideration should be given to monitoring high pressure areas as well as low pressure areas.

Opinions of probable project cost for the combined Phase I and Phase II main, storage and pumping improvements are summarized in Table 8-3 rounded to the nearest $10,000.

Table 8-3
Phase I and Phase II Recommended Improvements

Description		Units	Project Cost
Total Phase I Main Extensions (See Table 8-2)		Various	$11,010,000
Water main from Northeast PS to Vine St. Reservoir		26,800 ft	$13,020,000
All Other Phase II Main Extensions		Various	$36,780,000
Subtotal Phase I and Phase II Main Extensions			**$60,810,000**
Total Phase I Facilities Improvements (See Table 8-2)		Various	$1,340,000
Total Phase I Fire Flow Improvements (See Table 8-1)		Various	$460,000
New 20 mgd pump at Vine PS to Southeast SL		20 mgd	$1,000,000
New A St. Satellite Pumping Station to Low SL		10 mgd	$2,300,000
New WTP High Service Pump No. 13		20 mgd	$1,000,000
Pressure Monitoring Stations		5	$100,000
Total Phase I and Phase II Improvements			**$67,010,000**

(1) All project costs are reported in year 2007 dollars, and include a 20 percent contingency, plus a 20 percent allowance for engineering, legal, and administrative costs.

8.4.2 Phase III – Mid-term Improvements (by Year 2032)

Phase III recommended improvements will provide service to the limits of Tier I – Priority C development area. About $32 million of total piping improvements are recommended as Phase III (by year 2032). This equates to an average of about $2.5 million per year of required CIP funds for Phase III pipeline extensions. Recommended pumping and storage facilities to serve the year 2032 service limit are summarized below:

- Vine Southeast Pumping Station. Replace one 10 mgd pump at the Vine Southeast Pumping Station with a unit rated 20 mgd. Review of the pumping station design indicates that the station is designed to accommodate 20 mgd pumping units.

- Pioneers Pumping Station. Add fourth pumping unit at the Pioneer's Pumping Station rated at 5 mgd. This will increase the firm pumping capacity of the Pioneers Pumping Station to 10 mgd.

- WTP High Service Pumping. Additional High Service Pumping at WTP. Two additional pumps rated at 20 mgd and 350 for head should be installed by the year 2032. One of these pumps should replace the existing Pump No. 10 which has lower discharge head characteristics than the other high service pumps. The addition of a new Pump No. 14 will require the construction of a new pumping station at the WTP. This station has been planned to deliver to the distribution system the ultimate planned capacity of the WTP of 210 mgd. The new pumping station should contain space for three pumping units sized for 20 mgd each.
- New Yankee Hill Pumping Station: Construct new pumping station to replace the existing Cheney Pumping Station, to provide service to the Cheney Booster District. The pumping station should be designed for three pumps with a total installed capacity of 12 mgd and a firm capacity of 8 mgd to meet year 2057 demands. However, initially only two pumps would need to be installed to meet demands through year 2032.
- Additional Northeast Reservoir Storage Capacity. Additional storage capacity is not required for peak demands, but will allow for greater flexibility in terms of system operation, reliability, and in case of emergency. Space exists on site for to add 10 MG of additional storage. Additional property may need to be acquired for further storage capacity in the future.
- New Saltillo Reservoir. This 4 MG storage facility will help to meet maximum hour demands in the High Service Level and should have an overflow elevation of 1420. The reservoir will also provide needed additional reliability and redundancy for the High Service Level.
- New Southwest Storage Reservoir. This 5 MG storage facility will help meet maximum hour demand in the southern portion of the Belmont Service Level and should have an overflow elevation of 1400.
- New Northwest Storage Reservoir. This 1 MG storage facility will help to meet maximum hour demands and provide reliability for the Northwest Booster District. This facility should have an overflow elevation of 1460 and be located on high ground elevations of 1320 east of NW 12th Street.

Opinion of probable project cost for Phase III main, storage, and pumping improvements are summarized Table 8-4, rounded to the nearest $10,000.

Table 8-4
Phase III Recommended Improvements

Description		Units	Project Cost
Replace Pump SE1 at Vine Southeast Pumping Station		20 mgd	$1,000,000
Add Pump No. 4 at Pioneers Pumping Station		4 mgd	$200,000
Replace Pump No. 10 at WTP		20 mgd	$1,000,000
Construct New High Service Pumping Station and add Pump No. 14 (include space for three units)		20 mgd	$4,600,000
Yankee Hill Pumping Station		8 mgd	$1,840,000
Additional Northeast Storage Capacity (buried below-grade)		10 MG	$15,000,000
Saltillo Reservoir for High SL (above-grade)		4 MG	$4,000,000
Southwest Reservoir for Belmont SL (above-grade)		5 MG	$5,000,000
Northwest Reservoir for Northwest SL (elevated)		1 MG	$2,000,000
All Phase III Main Extensions		Various	$31,600,000
Total Phase III Improvements			**$66,240,000**

(1) All project costs are reported in year 2007 dollars, and include an 20 percent contingency, plus a 20 percent allowance for engineering, legal, and administrative costs.

8.4.3 Summary of Phased Improvements

The project costs for Phase I, II, and III recommended improvements are summarized in Table 8-5.

Major transmission main routing and sizing, storage reservoir location and sizing, and pumping station location were conceptualized for Phase IV improvements. Project costs were developed for major transmission mains for Phase IV, but not for pumping and storage facilities. The required capacities of future Phase IV pumping stations and storage reservoirs should be evaluated in greater detail in future master planning efforts.

Table 8-5
Summary Recommended Improvements

Description	Project Cost by Phase		
	Phase I Immediate Improvements	Phase II Short-term By Year 2019	Phase III Mid-term By Year 2032
Fire Flow Improvements (see table 8-1)	$460,000		
3.6 mgd Booster Pumping Station at I-80 west of N 56th St(3)	$1,290,000		
Control Valve in Pioneers Pumping Station	$50,000		
All Phase I Main Extensions	$11,010,000		
New 20 mgd pump at Vine PS to Southeast SL		$1,000,000	
New 10 mgd "A" Satellite Pumping Station to Low SL		$2,300,000	
New 20 mgd WTP High Service Pump No. 13		$1,000,000	
Pressure Monitoring Stations		$100,000	
All Phase II Main Extensions		$49,800,000	
Replace Pump SE1 at Vine Southeast Pumping Station with 20 mgd Pump			$1,000,000
Add 5 mgd Pump No. 4 at Pioneers Pumping Station			$200,000
Replace Pump No. 10 at WTP with 20 mgd Pump			$1,000,000
Construct New High Service Pumping Station and add 20 mgd Pump No. 14 (include space for three units)			$4,600,000
8.0 mgd Yankee Hill Pumping Station(3)			$1,840,000
Additional Northeast Storage Capacity (10 MG buried below-grade)			$15,000,000
Saltillo Reservoir for High SL (4 MG above-grade)			$4,000,000
Southwest Reservoir for Belmont SL (5 MG above-grade)			$5,000,000
Northwest Reservoir for Northwest SL (1 MG elevated)			$2,000,000
All Phase III Main Extensions			$31,600,000
Total by Phase	**$12,810,000**	**$54,200,000**	**$66,240,000**

(1) Annual main replacement and other facility rehabilitation projects are not included in this table.
(2) All project costs are reported in year 2007 dollars, and include a 20 percent contingency, plus a 20 percent allowance for engineering, legal, and administrative costs.
(3) Reported pumping station capacities are firm capacity recommendation.

8.5 Annual Investment for Main Extensions

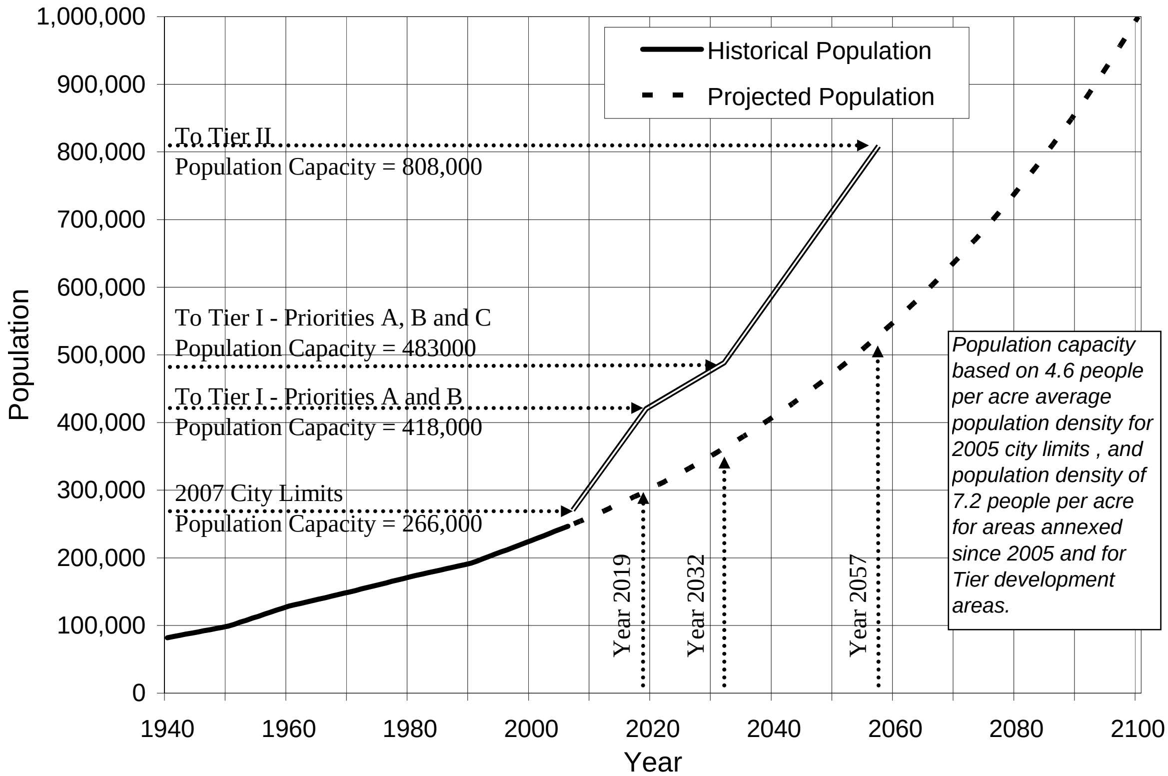
Excluding all future 12-inch lines, about $61 million of total piping improvements are recommended in Phase I and II (by year 2019). Only an additional $32 million of piping improvements are then recommended in Phase III (by 2032). Based on a total of $61 million of piping improvements over the next 12 years, about $5.0 million would be required in the CIP on an annual basis. After the initial 12-years, the CIP requirement would then decline to about $2.4 million per year.

There are two main reasons for the larger cost of improvement pipes for year 2019. First, the growth area defined by Tier 1 - Priority A and B (about 28 square miles) is larger than the growth area defined by Tier 1 - Priority C (about 14 square miles). Second, expansion into the growth areas often requires large diameter mains be constructed in Tier 1 Priority A and B by year 2019, in order to serve the Tier 1 Priority C areas in later years.

A review of the estimated year 2005 population and the year 2005 corporate limits shows that the population density within the city limits of the City of Lincoln was about 4.6 people per acre, or about 2,900 people per square mile. According to the City of Lincoln-Lancaster County Planning Department, a development density of about 7.2 people per acre, or about 4,600 people per square mile, is anticipated for the development tiers. Based on 7.2 people per acre, the required amount of extra land required to support the projected year 2019 population growth of 56,000 (year 2005 to year 2019) would be about 12 square miles, or slightly less than 1 square mile per year. Since 2005, about 5 square miles of land has been annexed into the City; and the 2030 City of Lincoln/Lancaster County Comprehensive Plan (adopted November 2006) provides for about an additional 33 square miles of land to be made available for development by year 2019 (Tier I – Priority A and Priority B).

The total area annexed since 2005 plus the Tier I – Priorities A and B areas collectively provide a little less than three times the area required to support projected growth through year 2019; based on the maximum anticipated population densities. Figure 8-1 shows the population capacity for the tier areas compared to the projected population for the design years used in this report. It shows that more land will be made available for development in the coming years than is required for the projected population. This excess of land provides for flexibility in the location of development, but may also commit the LWS to construction of transmission system improvements for potentially scattered development.

Figure 8-1: Projected Population and Tier Population Capacities



A review of the recommended improvements required to provide service to the entire Tier I growth area (including Priorities A, B, and C) reveals that on average, about $2.3 million is required to provide distribution system gridding and transmission mains (excluding all 12-inch and smaller pipes) for every square mile of newly developed area. Based on a land requirement of 1 square mile per year to support growth, about $2.2 million per year would be required in the early years to construct distribution system gridding and transmission mains. However, as indicated above, if scattered development occurs, and it is necessary to construct transmission main improvements to serve the entire Tier I – Priority A and B areas by 2019, the annual CIP cost requirement for main extensions would be closer to $5.0 million per year.

8.6 Annual Investment for Main Replacement

The main replacement program was reviewed with the following goals:

- Benchmark the progress of the existing program.

- Update maps to correlate repair and/or maintenance activities with replacement history.
- Establish the current state of GIS and provide recommendations for improvements to the database.
- Determine base line data for evaluating water main life cycle cost analysis and create replacement projection graph for distribution mains.

The existing 6-year capital improvement plan for LWS includes $2.75 million for main replacement and rehabilitation. Assuming a 100 year service life for water mains, one percent of the system should be renewed every year to prevent the system from deteriorating. This level of funding translates into $6.92 million per year, which is approximately 2.5 times the current budget.

There is a need for an increase in the pipe replacement program budget in order to preserve the distribution system asset value. However, such a need must be assessed in the broader context of other priorities. Consideration should be given to developing a comprehensive "asset management plan" to establish future fiscal needs for preservation of LWS assets.

There is no formal replacement criteria established to identify which mains should be replaced. Consideration should be given to conducting a more detailed pipeline replacement plan using a matrix rating system to prioritize mains for improvement. Most of the targeted mains are replaced by open cut methods. Consideration should be given to replacement by trenchless techniques as well as rehabilitation of existing lines.

Consideration should also be given to developing a pipeline inspection program for large diameter mains to assess their condition and conduct proactive maintenance if required, to reduce the risk of future catastrophic failures. Future testing should identify the condition of the transmission main from the water treatment plant to the 51st Reservoir because it is the oldest of the transmission mains from the water treatment plant into Lincoln. More detailed information of the condition of the main would be useful in identifying whether the main should be abandoned or rehabilitated in the future, and allow for any necessary refinements to recommended long range transmission and pumping improvements from the water treatment plant to Lincoln.

BLACK AND VEATCH CORPORATION
Lincoln Water System **B & V Project: 148582.0100**
2007 Facilities Master Plan Update **December 12, 2007**
Fire Flow Deficiency Analyses

To: Project File
Prepared By: James Maher, Jerry Edwards
Reviewed By: Andrew Hansen

Purpose

This memorandum documents the methodology for performing an analysis of the fire flow requirements, the ability of the Lincoln Water System (LWS) distribution system to meet these requirements, and the results and recommendations resulting from those analyses; as described in Task Series 600 – Fire Flow Simulation of the Facilities Master Plan Update for the Lincoln Water System

Fire Flow Goals

In addition to routinely serving water for residential, commercial, industrial and institutional uses, municipal water systems are generally expected to deliver adequate quantities of water for fire fighting purposes. Besides the direct benefit of fire protection, a capable water system provides the indirect benefit of improving the fire insurance rating and thereby lowering insurance premiums for a community. Insurance ratings are determined by the Insurance Services Office (ISO) based on system performance and characteristics.

Part of an ISO evaluation consists of determining needed and available fire flows at various locations throughout a water utility. The needed fire flow is calculated based on the size, construction, occupancy, and exposure of each building or complex. Needed fire flows can range from 500 gpm to 12,000 gpm. For insurance rating purposes, 3,500 gpm is the maximum fire flow required to be supplied by a municipal water system. Fire flow requirements in excess of 3,500 gpm which cannot be met by the water system may affect the rating of the individual building. However, the overall municipal rating determined by ISO will not be affected. This limit is rooted in practical considerations, which include avoiding the extra capital cost of overbuilding to meet needs of a low probability occurrence that could be avoided by actions of a few property owners, and avoiding deterioration of water quality in the distribution system that would result from increased water ages in oversized pipelines and reservoirs.

For the purpose of this study, fire flow goals were developed based on the Insurance Services Office (ISO) criteria for needed municipal fire flows. Rather than identifying needed fire flow for current buildings on individual parcels, municipal fire flow goals were established based on the land use designations from City of Lincoln GIS zoning data. In general, conservative values were chosen for the initial fire flow goals in order to identify all locations which may have potential deficiencies. Table 1 shows the land use designations with the fire flow goal assigned for analysis purposes.

Table 1
Fire Flow Goals within Fire Protection Area

Zoning Designation[1–	Description	Fire Flow Goal, gpm
AG	Agriculture District	1,000
AGR	Agriculture Residential	1,000
B-1	Local Business District	2,000
B-2	Planned Neighborhood Business District	2,500
B-3	Commercial District	2,500
B-4	Lincoln Central Business District	3,500
B-5	Planned Regional Business District	3,500
H-1	Interstate Commercial District	2,500
H-2	Highway Business District	3,500
H-3	Highway Commercial District	3,500
H-4	General Commercial District	2,500
I-1	Industrial District	3,500
I-2	Industrial Park District	3,500
I-3	Employment Center District	3,500
O-1	Office District	2,500
O-2	Suburban Office District	2,500
O-3	Office Park District	2,500
P	Public Use District (Government, Institutional, Other)	3,500
R-1	Low Density Residential Development	1,000
R-2	Low Density Residential Development	1,000
R-3	Low Density Mixed Housing Development	1,000
R-4	Low Density Residential Development	1,000
R-5	Medium Density Residential Development	2,000
R-6	Medium Density Residential Development	2,000
R-7	High Density Residential Development	2,500
R-8	High Density Residential Development	2,500
R-T	Residential	2,000
OPEN SPACE	Public Open Space	1,000
PARK	Public Park	1,000
SCHOOL	School	3,500
UNIVERSITY	University	3,500
AIRPORT	Airport	3,500

(1) Zoning provided Lincoln and Lancaster County Planning Department

Fire Flow Hydraulic Simulation

The fire flow standards set by the ISO require a minimum residual water pressure of 20 psi during a fire. Residual pressure, in this instance, is defined as the pressure inside the pipeline system near the points at which hydrant flows are taking place. From a fire fighting perspective, the principle reason for a required minimum residual pressure of 20 psi is that this pressure is sufficient to overcome the friction losses in the hydrant branch, hydrant, and suction hose with some pressure remaining at the fire pump. From a water quality perspective, the 20 psi residual is consistent with AWWA requirements for minimum system pressure to prevent backflow contamination. The *Distribution System Requirements for Fire Protection* manual (AWWA M31) indicates that the system should be capable of supplying the required fire flow during the maximum day demand condition.

The base year maximum day scenario of the hydraulic computer model was used to determine fire flow capacity throughout the service area. The demand during this scenario was 104.7 mgd and the system operation was similar to operations during the base year maximum day analysis. The hydraulic analysis included the following:

- Sufficient pumps were turned on at the Northwest Booster Pumping Station into the Northwest booster district to ensure that fire flows could be delivered without dropping the station discharge pressure below the normal 80 psi set-point.
- The Cheney Reservoir was included in the analysis. This tank is expected to be in service in early 2008.
- Tank levels were set to the higher level of two possible conditions - half full or 20 feet depleted

The fire flow capacity was determined by the hydraulic model for all model nodes with the constraint of maintaining a residual pressure of 20 psi at the junction location. It is important to recognize that fire hydrants are not included in the hydraulic model, and that the flows were calculated for each junction in the hydraulic mode which generally represent tees, crosses, and changes in pipe diameter.

Fire Flow Potential Deficiencies

Using GIS techniques, the available fire flow at each node as calculated by the model was used to create a surface of available flows. This surface was then intersected with the surface of fire flow goals based on zoning. Areas of potential deficiencies were identified where the calculated available fire flow was less than the needed fire flow. Following quality control checking, including a thorough review of the model results, potential fire flow deficient areas were identified as shown on Figure 1.

It is important to recognize that an area of potential fire flow deficiency does not necessarily mean that a deficiency exists, but that the system cannot deliver the established fire flow goal used for the analysis. Actual needed fire flows may be less than the established goal due to specific occupancy, exposure, or construction; or the building may be provided with a sprinkler system which would significantly reduce the needed fire flow. For example, this report used a minimum residential fire flow goal of 1000 gpm to identify potentially deficient areas, however needed fire flows for some residential areas may be lower than this goal as shown in Table 2. The use of these conservative numbers is considered appropriate for a planning level review, and to ensure the greatest identification of potential deficiencies.

Table 2
Needed Fire Flow for One-family and Two-family Dwellings (1)

Distance Between Buildings (ft)	Needed Fire Flow (gpm)
Over 100	500
31 – 100	750
11 – 30	1000
Less than 11	1500

(1) Dwellings not to exceed two stories in height

The areas of potential deficiencies were grouped into three categories - industrial/commercial, school, and residential – and improvement considerations were evaluated for the potential deficiencies. Specific improvement considerations were not identified for all potential deficiencies in the industrial/commercial and school categories. Many of these areas were determined to have relatively good available fire flows, but were still below the fire flow goal. Additional investigations should be conducted to verify the actual fire flow goals for the identified potentially deficient areas. It is likely that some of the fire flow goals exceed the needed fire flow. It is also important to recognize that the presence of automatic sprinkler systems will greatly reduce the needed fire flow. A discussion of needed fire flows for structures with automatic sprinkler systems is provided in the next section.

The potentially deficient areas are summarized in the following Table 3, Table 4, and Table 5 by category. Improvement considerations are provided for all of the residential category, and many of the industrial/commercial category potential deficiencies. However, unless a specific “RECOMMENDED IMPROVEMENT” is identified in the tables, additional investigations should be conducted to evaluate whether the fire flow goals might be greater than the actual needed fire flow. The suggested improvement considerations would then require implementation only if additional evaluations verify that the area is deficient.

Table 3
Potential Fire Flow Deficiencies – Residential Zoning Category

Map ID	Zoning	Avail. FF (gpm)	FF Goal (gpm)	Comments [(1)]
R1	AG	500	1000	W 58th & Partridge, dead-end 6-inch RECOMMENDED IMPROVEMENT - Construct loop main
R2a	R-2	600	1000	20th St, C to D, area with 4-inch mains RECOMMENDED IMPROVEMENT - Replace 4-inch mains on C and D with larger
R2b	R-6	1600	2000	22nd between B and D, E and F between 20th and 22nd Service by 6-inch and 4-inch mains Consider replacing 4-inch mains with larger if necessary
R3	R-3	600	1000	Northwest of NW 56th and Aurora, area with single 6-inch feed RECOMMEDED IMPROVEMENT -Complete connection on Aurora
R4a	R-5	600	2000	South of Wesleyan University, east of 53rd 6-inch dead-end mains at PZ boundary provide limited flow RECOMMENDED IMPROVEMENT - Complete looping on 53rd to increase fire flows
R4b	R-5	1600	2000	East side of Wesleyan University along 53rd, non-residential area Service by 6-inch mains, Consider new 6-inch feed at 53rd
R5	R-2	700	1000	Southeast of Van Dorn & 56th, area with single 6-inch feed Consider constructing loop main if necessary
R6	R-2	900	1000	58th and Cedarwood, circle drive with single 6-inch feed Consider constructing loop main if necessary
R7	R-4	900	1000	23rd and Potter, area with 4-inch mains Consider replacing 4-inch mains with larger if necessary
R8	R-6	1000	2000	19th and Garfield, area with 4-inch mains Consider replacing 4-inch mains with larger if necessary
R9	R-5	1000	2000	15th and Mulberry, 16th & Harwood, area with 4-inch mains Consider replacing 4-inch mains with larger if necessary
R10	R-5	1200	2000	O Street, 28th to 30th, mains at PZ boundary provide limited flow Consider replacing 4-inch on 30th, J to N, with 8-inch if necessary
R11	R-5	1500	2000	P St. and C St. – 24th to 26th, 6-inch and 4-inch mains Consider additional connection between O and P, at 30th
R12	R-5	1500	2000	B St. and C St. – 24th to 26th, 6-inch and 4-inch mains Consider replacing 4-inch mains with larger if necessary
R13	R-6	1500	2000	46th and Stockwell, area with 4-inch mains Consider replacing 4-inch mains with larger if necessary
R14	R-6	1600	2000	19th and 20th south of J Street, 6"/4" loop piping Consider replacing 4-inch mains with larger if necessary
R15	R-5	1700	2000	Baldwin and Huntington, 35th to 42nd Long reach of 6-inch not interconnected at intermediate streets Consider constructing 6-inch mains for addition looping

[(1)] Unless "RECOMMENDED IMPROVEMENT" is identified, additional investigations should be conducted to evaluate whether the fire flow goals might be greater than the actual needed fire flow.

Table 4
Potential Fire Flow Deficiencies – Industrial/Commercial Zoning Category

Map ID	Zoning	Avail. FF (gpm)	FF Goal (gpm)	Comments (1,2)
IC1	O-3	1000	2500	Southwest of Fletcher and NW 12th, new development area Dead-end 6-inch feed into area RECOMMENDED IMPROVEMENT – Construct second feed
IC2	B-3	1800	2500	Touzalin Ave and 61st, north of Havelok, Upper end of distribution mains
IC3	H-4	2000	2500	Southeast of 27th & Pine Lake, new commercial area located at PZ boundary
IC4	I-1	1000	3500	5th and 6th at K and J, large loop with 4-inch and 6-inch Consider upsizing mains if necessary
IC5	I-1	1500	3500	Irving, west of 27th, dead-end 6-inch feed at PZ boundary Consider PRV for fire protection at 23rd and Irving in necessary
IC6	I-1	1700	3500	Yalande Ave, west of N 19th, dead-end 8-inch feed at PZ boundary Consider PRV for fire protection at Yolande & Cornhusker if necessary
IC7	I-1	2000	3500	Adams and Cleveland, 36th to 38th, 6-inch gridding
IC8	I-1	2000	3500	3rd from Rose to A; Rose and Peach at 6th, 6-inch gridding
IC9	I-1	2000	3500	Industrial Areas at Van Dorn & 4th, 8-inch gridding Consider PRV for fire protection at Park & Hatch if necessary
IC10 IC11 IC12	I-1	2000	3500	Airpark industrial areas Consider upsizing mains if necessary
IC13	P	2000	3500	58th and 59th, south of Huntington, 4-inch on Huntington with private 10-inch main in area Consider upsize of 4-inch if necessary
IC14	I-1	2500	3500	27th north of Fair, and Leighton east of 27th
IC15	I-1	2600	3500	P St, west of Sun Valley
IC16	I-1	2600	3500	Seward 66th to 69th
IC17	I-1	2800	3500	Seward and Toulazin
IC18	H-3	3000	3500	O St, west of Capital Beach

(1) Unless “RECOMMENDED IMPROVEMENT” is identified, additional investigations should be conducted to evaluate whether the fire flow goals might be greater than the actual needed fire flow.
(2) Improvement considerations may not be provided for all potential deficiencies and should be evaluated further if additional evaluations verify that area is deficient.

Table 5
Potential Fire Flow Deficient Areas – School Category

Map ID	Zoning	Avail. FF (gpm)	FF Goal (gpm)	Comments [1,2]
S1	UNIVERSITY	1000	3500	University parking structure and buildings just south of the university bioscience greenhouses
S2	SCHOOL	1700	3500	Saratoga Elementary
S3	SCHOOL	2000	3500	Lincoln Northeast High School
S4	SCHOOL	2400	3500	Norwood Park Elementary
S5	SCHOOL	2500	3500	Lakeview Elementary School
S6	SCHOOL	2500	3500	Sheridan Elementary School
S7	SCHOOL	2500	3500	Lincoln Southeast High School
S8	UNIVERSITY	2500	3500	Southeast Community College, northern end
S9	SCHOOL	2700	3500	Pound Middle School
S10	SCHOOL	3000	3500	Randolph Elementary School

(1) No recommended improvements are identified for school and university zoning. Additional investigations should be conducted to evaluate whether the fire flow goals might be greater than the actual needed fire flow, especially with regard to presence of automatic sprinkler system. Improvement considerations should be evaluated further if additional evaluations verify that area is deficient.

Automatic Fire Sprinkler Systems

The ISO fire suppression rating schedule identifies needed fire flows for buildings without automatic fire sprinkler systems. The presence of an automatic sprinkler system in a structure will reduce the needed fire flow for that structure. This section provides an overview of needed fire flows for buildings with automatic sprinkler systems.

The determination of the required flows for a sprinklered building can be calculated based on information provided by the National Fire Protection Association (NFPA) in *Standard No. 13 – Standards for Installation of Sprinkler Systems*. Required fire flows for sprinklered buildings include the flow required for the sprinklers plus a hose allowance, or 500 gpm, whichever is greater. When designing sprinkler systems, structures are classified by "hazard group". Average commercial occupancies such as retail stores, offices, hotels, and institutional buildings will usually be in the category of ordinary hazard group 1. Warehouses and manufacturing occupancies will be in the category of ordinary hazard group 2 or 3. Occupancies in which there are highly flammable products or processes in large quantities will be classified as extra hazard. The water distribution system must be able to provide the required sprinkler flow (based on area of coverage and occupancy), plus a hose allowance to account for simultaneous operation of inside or outside hose streams. Typical needed fire flows for buildings with automatic fire sprinkler systems are summarized in Table 6.

Table 6
Needed Fire Flows for Buildings with Automatic Sprinkler Systems [1]

Hazard Classification	Maximum Sprinkler Flow (gpm) [2]	Hose Stream Allowance (gpm)	Total Needed Fire Flow (gpm)
Light	200	100	300
Ordinary Group 1	400	250	650
Ordinary Group 2	600	250	850
Ordinary Group 3	750	500	1250
Hazard Group 1	1200	500	1700
Hazard Group 2	1500	1000	2500

(1) Based on NFPA Standard 13. Information in this table is not the complete and official position of the NFPA which is represented only by the standard in its entirety.
(2) Increased flows may be required for buildings where rack storage is present.

Generally, sprinkler systems are designed as a “pipe schedule system”, or a “hydraulically calculated system”. For a pipe schedule system, the required fire flow should be available at the base of the sprinkler riser at a pressure equivalent to 15 psi at the highest sprinkler. Hydraulically calculated systems require a minimum pressure of 7 psi at every sprinkler. Accounting for friction losses from the main to the base of the riser, the distribution system should be able to deliver the total needed fire flow for a sprinkler building of two stories or less, at a minimum pressure of 20 psi. Taller buildings would require greater pressure at the water main and hydraulically calculated designs may require greater pressures.

Conclusions and Recommendations

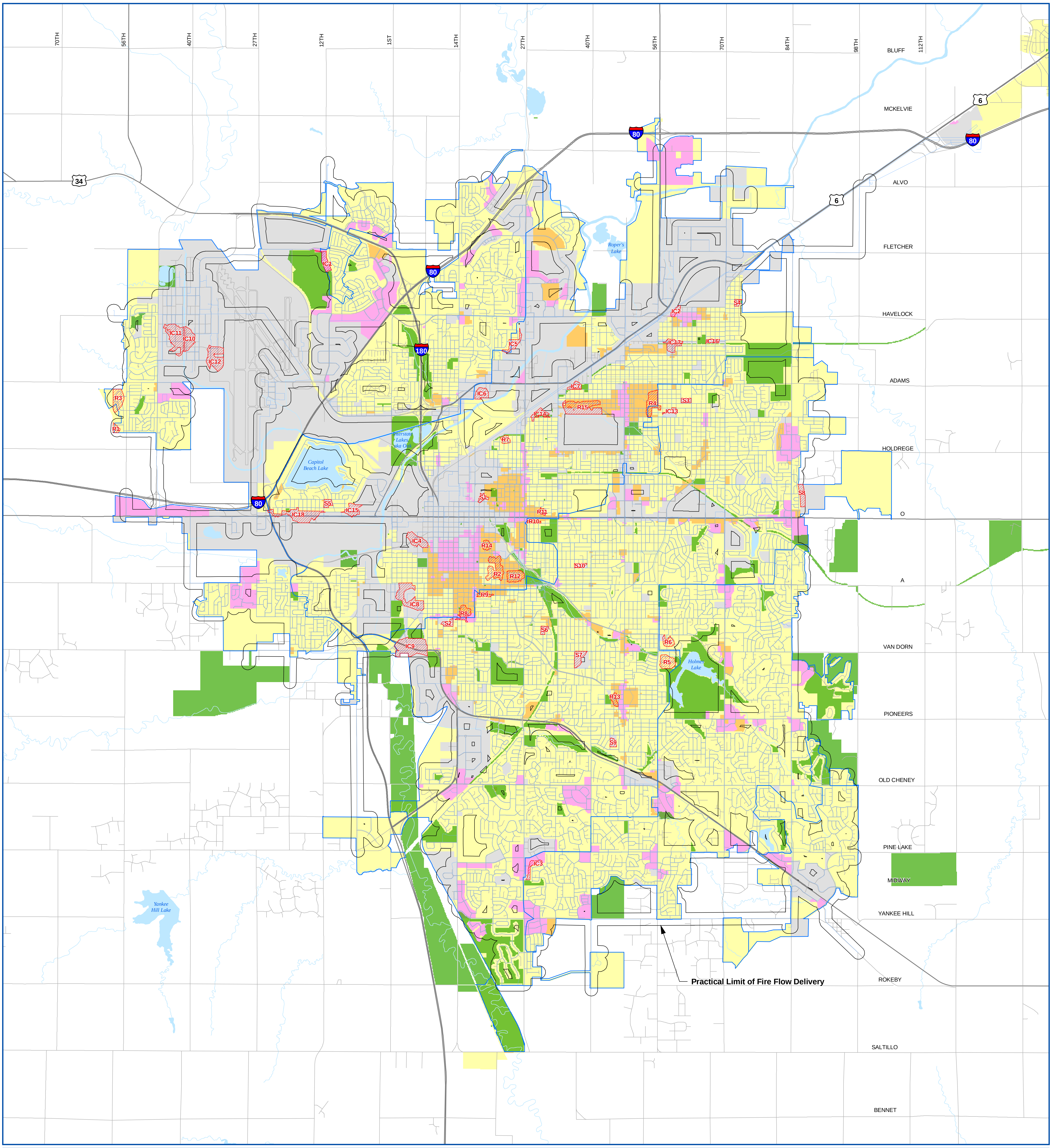
As stated above, it is important to recognize that an area of potential fire flow deficiency does not necessarily mean that a deficiency exists, but that the system cannot deliver the established fire goal used for the analysis. Actual needed fire flows may be less than the established goal.

It is equally important to recognize that not all fire flow deficiencies may be identified by the analysis conducted for this study. The model does not include junctions at all fire hydrants. Therefore, for example, a long reach of 4-inch main may have a potentially deficient hydrant in the center of the reach but have adequate flows on each end where there are model junctions.

Only six specific recommended improvements are identified to correct potential fire flow deficiencies in five areas. Actual fire flow goals should be verified for other potentially deficient areas before implementing improvements. Fire flow improvement capital costs including a 20 percent contingency and 20 allowance for engineering, legal, and administrative costs are based on a unit capital cost of $120 per foot in developed areas or very short lengths, and $80 per foot in undeveloped areas. The recommended fire flow improvements are summarized in Table 7.

Table 7
Recommended Improvements for Fire Flow Deficiencies

Map ID	Comment		Length (ft)	Capital Cost
R1	Construct 6-inch on Partridge between W 58th and W 57th		590	$65,000
R2b	Replace 4-inch on D between 18th and 20th with 6-inch		1000	$120,000
R3	Complete 6-inch connection on Aurora at W 56th		100	$12,000
R4a	Complete looping on 53rd, north of Huntington (May not be required if building is sprinklered)		600	$72,000
R4a	Construct 6-inch looped main at PZ boundary, 53rd between Leighton and Huntington		700	$84,000
IC1	Construct 8-inch looped main at PZ boundary, Fletcher between W 15th and W 12th		900	$108,000
	TOTAL FIRE FLOW IMPROVEMENTS			$461,000



Lincoln Water System
2007 Facilities Master Plan Update

POTENTIAL FIRE FLOW DEFICINECIE

Figure 1

Legend

R1 Potential Deficiency and ID

Service Level

Existing Pipeline

Minimum Fire Flow Service Criteria

- 3500 gpm
- 2500 gpm
- 2000 gpm
- 1000 gpm
- 0 gpm - Parks

0 0.2 0.4 0.8 1.2 1.6 Miles

1 inch equals 5,000 feet

***Note:** Practical limit of fire flow delivery defined to be within 500 feet of existing water distribution main.*

CITY OF LINCOLN NEBRASKA

BLACK & VEATCH Building a world of difference

Data source: City of Lincoln, B&V
Path: C:\Projects\Lincoln\11GIS\01MapDocuments\Fire_Flow.mxd Date Printed: December 12, 2007 Map By: B. Lendt

BLACK AND VEATCH CORPORATION
Lincoln Water System **B & V Project: 148582**
2007 Facilities Master Plan Update **February 12, 2008**
Memorandum – Water Age Operational Validation

To: Project File
Prepared By: James Maher
Reviewed By: Jerry Edwards

Purpose

The purpose of this memorandum is to document the existing conditions water age analyses and outline the methodology used to set the operational controls in these analyses. Due to the nature of the water age calculations in the hydraulic modeling software, the operational controls and system demands will have a great impact on the relative values of residence times calculated at locations within the system. The basic concept when setting model controls and demands is to mimic system operations during specified demand conditions to the greatest extent possible.

In a distribution system, pump status, valve status, and reservoir fluctuation are all part of system operations over which an operator has real time control. Although some pump controls may be automated, such as lead or lag controls or on or off settings, ultimately an operator's experience takes precedence over such automation. In the modeling software (H2OMap Water), the ability for real time control is limited by the manner in which a water age extended period simulation is performed. After a simulation's time options (length and time-step) and operational controls are set and the "run" button is pushed, there is no way of interrupting the simulation to review instantaneous results at any given time and change operational parameters accordingly. This issue is further complicated by the fact that logic and rule-based controls set by the user must continuously be achieved in order for a hydraulic simulation to solve over the entire length of the simulation. These issues constrain the user to set operational controls that will always solve and must be maintained constant during the entire length of the simulation which, for a water age analysis, can be a relatively lengthy duration.

Model Scenario and Operational Control Validation

To determine control settings to simulate actual system operations and water movement over the course of a specified demand period, several considerations were evaluated and iterative changes were made to controls until the results closely resembled recorded data. The following key considerations were taken into account as criteria for setting the demand conditions and operational parameters (logical and rule-based controls) for the hydraulic model:

- The modeled system demands must resemble the recorded system demands during the duration of the simulation.

- Modeled reservoir water levels must remain within (or reasonably close to) the minimum and maximum recorded water levels
- The number of fill/draw cycles for reservoirs must closely resemble the actual number of fill/draw cycles recorded over the duration of the scenario.
- The modeled rate at which the reservoirs fill/draw should be consistent with the recorded rates.
- Modeled average flows from pump stations must be reasonably close to the average pump station flows recorded.
- Modeled pumping rates during pump station "on" status should be consistent with the recorded flow rates during the simulation.
- The modeled number of hours of pump station "on" status should be consistent with the recorded number of hours of pump station "on" status during the simulation time period.

Distribution System Demands

Two periods of demand were reviewed in the validation of operational controls used in the water age analyses. SCADA data for a two week period during January 2007 and a two week period during October 2007 were requested by B&V and provided by LWS. These data were used to develop a minimum month demand scenario and an average month demand scenario respectively. For each of these demand scenarios the recorded SCADA data was compiled and reviewed, and "synthetic" or typical, average diurnal demand patterns were developed for input into the hydraulic model scenarios. The ten days of weekday demands were averaged to represent an average weekday diurnal and the four days of weekend demands were averaged to represent an average weekend diurnal for each of the scenarios. Figure 1 shows the average weekday and weekend diurnals calculated from the January SCADA data and Figure 2 illustrates the average weekday and weekend diurnals calculated from the October data. In the model analysis, the average weekday demand pattern was repeated for five days, followed by two days of the average weekend demand, and this 7-day pattern was then repeated for six weeks.

It should be noted that the Southeast reservoir and pump station were offline for reservoir maintenance during the October data period. For general system operations this would be an abnormal condition and for the water age analyses the Southeast reservoir and pump station were set as operational in the model. The January data showed that there is typically an average difference of about 2 feet in the gradients between the Pine Lake Reservoir and the Southeast Reservoir and this was taken into account in the verification of the average month operations.

Figure 1 - LWS Average Diurnals, 1-10-2007 Through 1-23-2007

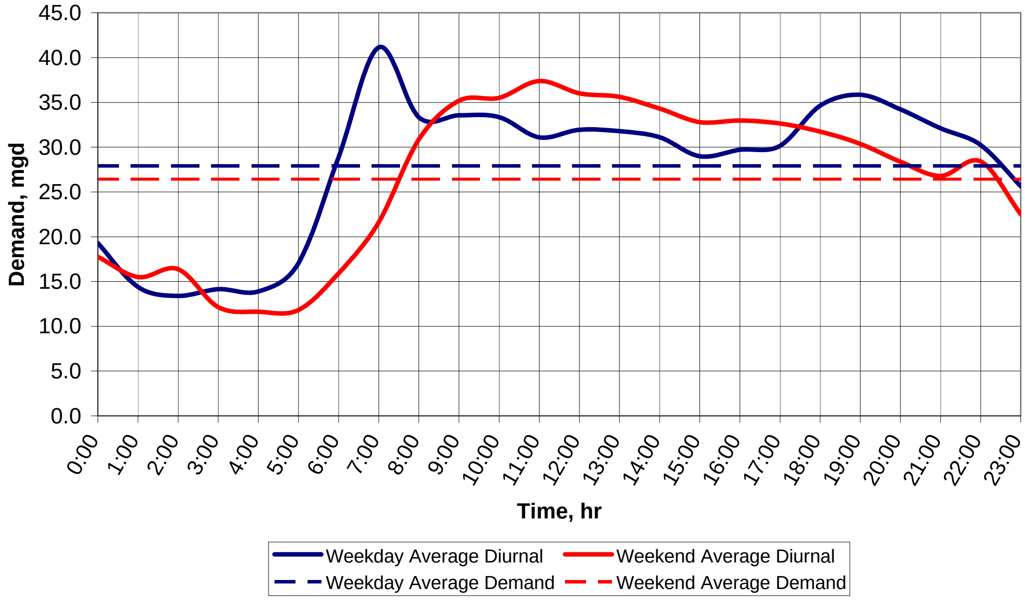
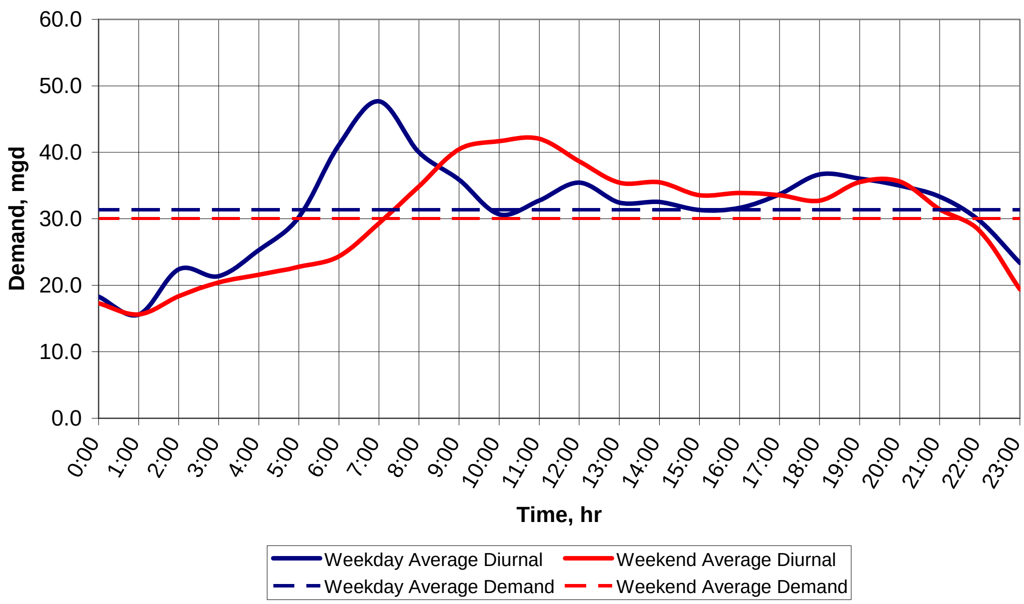


Figure 2 - LWS Average Diurnals, 10-10-2007 Through 10-23-2007



January 2007 Operational Validation

<u>Reservoir Fluctuations and Fill/Draw Characteristics:</u> A comprehensive review of the SCADA data was performed and graphs were developed to aid in the review of the results for the water age analyses. Each of these graphs illustrates reservoir levels during the period of the provided SCADA data and shows the minimum and maximum fluctuating ranges. During the operational control setting process these graphs were used to substantiate slight adjustments to controls in order to obtain greater accuracies in modeled operations. Attachment A and Attachment B show graphs of actual versus modeled water levels for the January, 2007 and the October, 2007 operational validation analyses. From these graphs it can be seen that reservoir fill/draw characteristics and fluctuation ranges from the model results closely resemble the recorded data. It is important to note that the modeled results will not be identical to the recorded results because of several factors, including but not limited to the following:

- An average system-wide diurnal was used for the entire system and individual Service Level actual demands may have been slightly greater or lesser than the resulting calculated demands.
- Some key data points were not recorded in SCADA for the time frame of these analyses. There is no meter to record the transfer flow from Northeast to 51st. Pumped flow from Vine to the Southeast Service Level, and transfer flows through control valves at Vine and Pine Lake Reservoir were not recorded, and as a result of these evaluations, they have since been corrected to record properly.
- Southeast Reservoir was offline for maintenance during the two weeks of recorded data. During typical operations this reservoir will be in service and the model simulation was developed with Southeast Reservoir included in order to represent more typical operating conditions.
- For the cases of the "A", Northeast, and the Vine Reservoirs, the hydraulic model was developed to simulate these facilities as one large storage facility with a diameter and sidewater depth resulting in volumes equivalent to the sum of the individual storage facility volumes at each of these locations. The software produces unstable results when two storage facilities are in extremely close hydraulic proximity to each other and a "sloshing" effect is introduced into the calculations.

Table 1 and Table 2 summarize the modeled results versus the recorded results in terms of minimum and maximum reservoir level ranges for each scenario. These tables show that in most cases the reservoir levels fall well within the ranges and when a deviation from the actual range occurs it is small enough to be deemed as insignificant.

Table 1 - Modeled Reservoir Level Results vs. Recorded Reservoir Level Results January 2007 Operational Validation

Facility	Recorded Maximum Level	Recorded Minimum Level	Modeled Maximum Level	Modeled Minimum Level	Maximum Level Criteria	Minimum Level Criteria
Air Park	91.5	72.0	88.4	70.0	Okay	2 ft Lower
Northwest	67.5	49.8	67.8	50.0	0.3 ft Higher	Okay
Pioneer	53.0	40.2	54.0	42.0	1 ft Higher	Okay
Vine	26.3	16.9	27.2	16.9	0.9 ft Higher	Okay
Pine Lake	60.8	48.2	58.9	48.0	Okay	0.1 ft Lower
Southeast	59.4	45.0	58.9	45.0	Okay	0 ft Lower
Yankee Hill	71.9	58.1	71.6	60.0	Okay	Okay
Northeast	15.9	11.0	15.6	10.9	Okay	0.1 ft Lower
51^{st}	13.3	8.8	13.3	8.9	Okay	Okay

Table 2 - Modeled Reservoir Level Results vs. Recorded Reservoir Level Results October 2007 Operational Validation

Facility	Recorded Maximum Level	Recorded Minimum Level	Modeled Maximum Level	Modeled Minimum Level	Maximum Level Criteria	Minimum Level Criteria
Air Park	91.8	77.1	92.4	76.6	0.6 ft Higher	0.5 ft Lower
Northwest	72.4	56.9	72.4	56.9	Okay	Okay
Pioneer	52.6	43.1	52.6	42.4	Okay	0.7 ft Lower
Vine	27.4	17.1	26.3	16.6	Okay	0.5 ft Lower
Pine Lake	62.1	47.7	62.0	50.1	Okay	Okay
Southeast (1)	0.0	0.0	60.0	46.5	60 ft Higher	Okay
Yankee Hill	71.7	63.7	71.9	63.7	0.1 ft Higher	0.1 ft Lower
Northeast	15.8	11.5	15.6	11.4	Okay	0.1 ft Lower
51st	13.3	10.0	13.5	10.0	0.2 ft Higher	Okay

(1) Southeast Reservoir was offline for maintenance during the two weeks of data. The model simulation was performed with Southeast reservoir online to represent typical operating conditions. The results are similar to January recorded data which indicate that this reservoir operates an average of 2 feet below the range of Pine Lake.

<u>Pump Station Flows:</u> Consistent with the methodology proposed in the model scenario and operational control validation section of this memorandum, the modeled flows were compared with the recorded flows from SCADA. These graphs are found in Attachment C and Attachment D of this memorandum and document the recorded hourly flows and the average flow over the entire two week periods for each scenario. The model results were also graphed along with the SCADA data and used in conjunction with the reservoir

fluctuation and fill/draw characteristics to adjust the model controls until reasonable results were achieved. The same factors that limit the ability to achieve identical results that applied to reservoir levels are also applicable to the modeled flows. As shown in Table 2 and Table 3 the average modeled flows closely resemble the average recorded flows during the length of the simulations. The inspection of the graphs in Attachment C and Attachment D also show that the number of hours and magnitudes of flows through pump stations during operating hours were also very similar in nature at the end of the iterative adjustments to the operational controls used in these analyses.

Table 2 - Average Recorded Flows vs. Average Modeled Flows
January 2007 Operational Validation

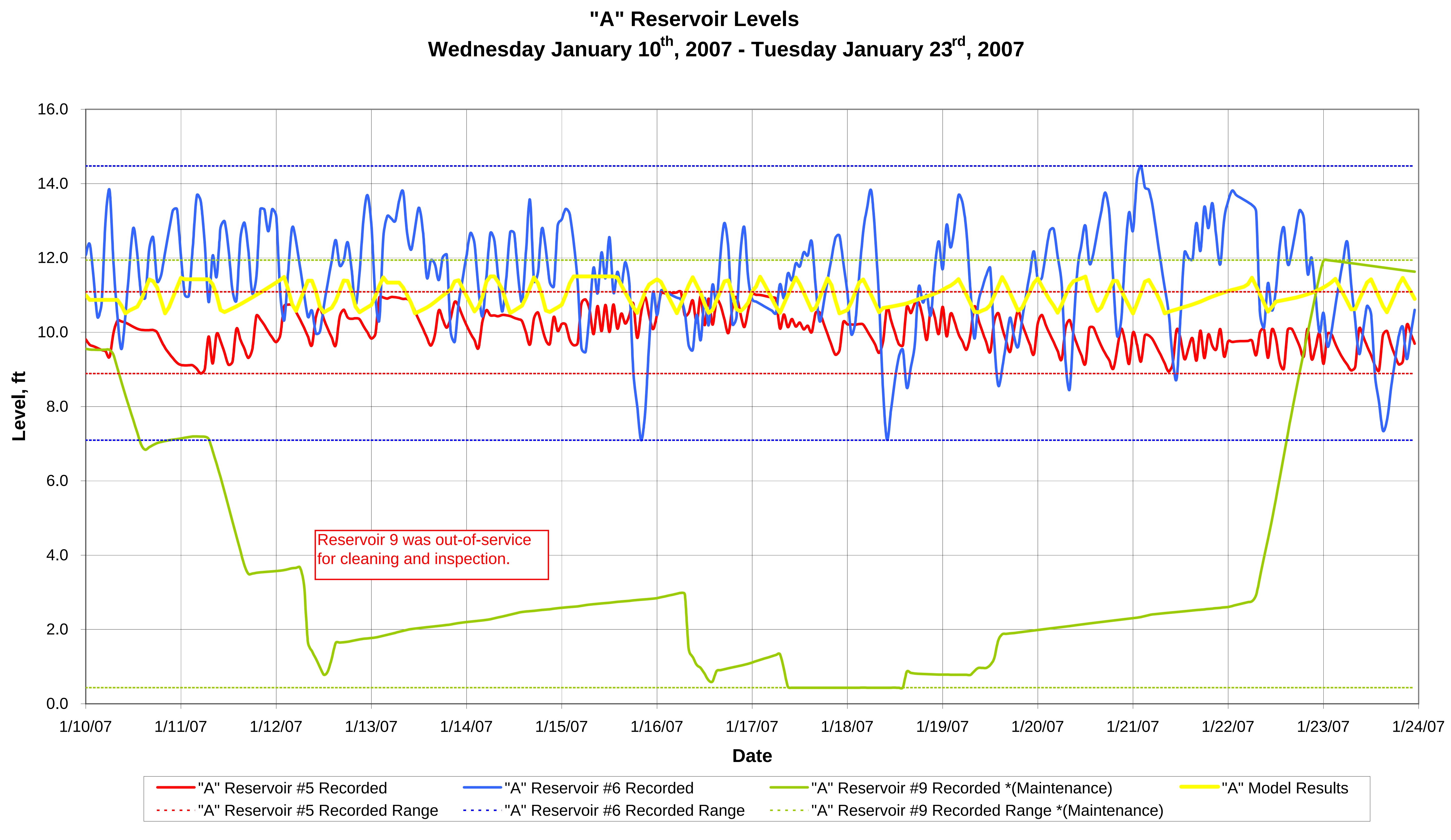
Facility	Average Recorded Flow Rate (mgd)	Average Modeled Flow Rate (mgd)	Difference	Percent Difference	Percent of Total Flow	Weighted Percent Difference
"A" Low Flow	7.62	7.84	-0.22	3%	15%	**0%**
"A" High Flow	6.49	4.85	1.64	25%	13%	**3%**
Belmont Flow	3.40	2.86	0.54	16%	7%	**1%**
51st Transfer	13.86	12.78	1.09	8%	28%	**2%**
51st Low Flow	11.97	14.41	-2.44	20%	24%	**5%**
Southeast Flow	0.67	1.12	-0.45	67%	1%	**1%**
Vine Flow	3.54	2.86	0.68	19%	7%	**1%**
Northeast Flow	1.51	1.19	0.32	21%	3%	**1%**
Pioneers Flow	1.29	1.20	0.09	7%	3%	**0%**

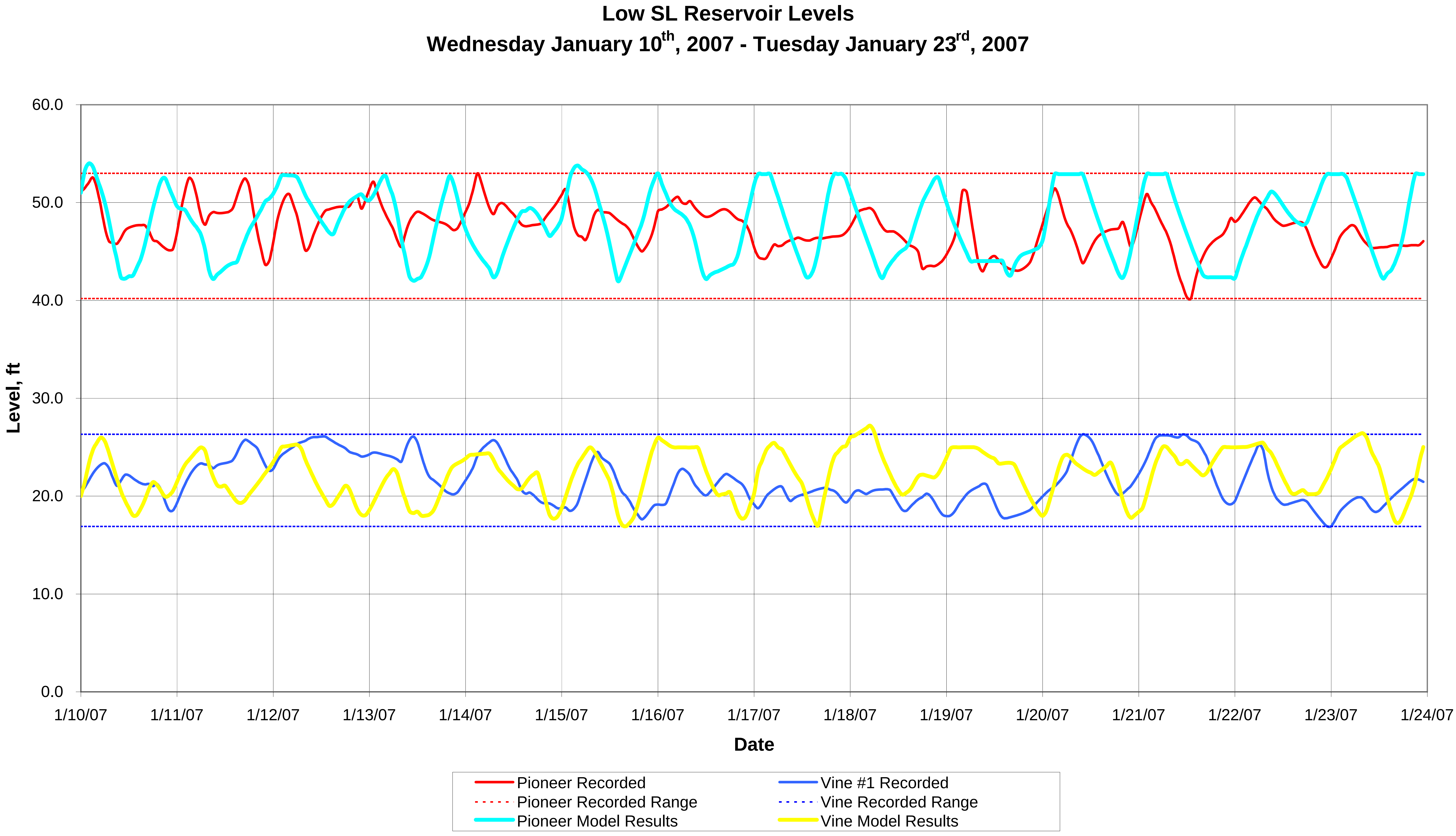
Table 3 - Average Recorded Flows vs. Average Modeled Flows January 2007 Operational Validation						
Facility	**Average Recorded Flow Rate (mgd)**	**Average Modeled Flow Rate (mgd)**	**Difference**	**Percent Difference**	**Percent of Total Flow**	**Weighted Percent Difference**
"A" Low Flow	8.03	8.45	-0.43	5%	15%	**1%**
"A" High Flow	9.15	8.86	0.29	3%	17%	**1%**
Belmont Flow	3.84	3.65	0.19	5%	7%	**0%**
51st Transfer	15.97	17.68	-1.71	11%	30%	**3%**
51st Low Flow	8.37	9.00	-0.63	8%	16%	**1%**
Southeast Flow(1)	0.00	1.71	-1.71	0%	0%	**0%**
Vine Flow	1.42	3.65	-2.23	158%	3%	**4%**
Vine Adjusted(1)	*3.12*	*3.65*	*-0.53*	*17%*	*6%*	***1%***
Northeast Flow	4.99	4.46	0.53	11%	10%	**1%**
Pioneers Flow	0.65	0.94	-0.29	45%	1%	**1%**

(1) Southeast Pump Station was not used during the period of recorded data because of the maintenance of Southeast Reservoir. Because less supply was required in the High Service Level under this condition, the '***Vine Adjusted'*** row reports the variances from normal operations if this had not been the case.

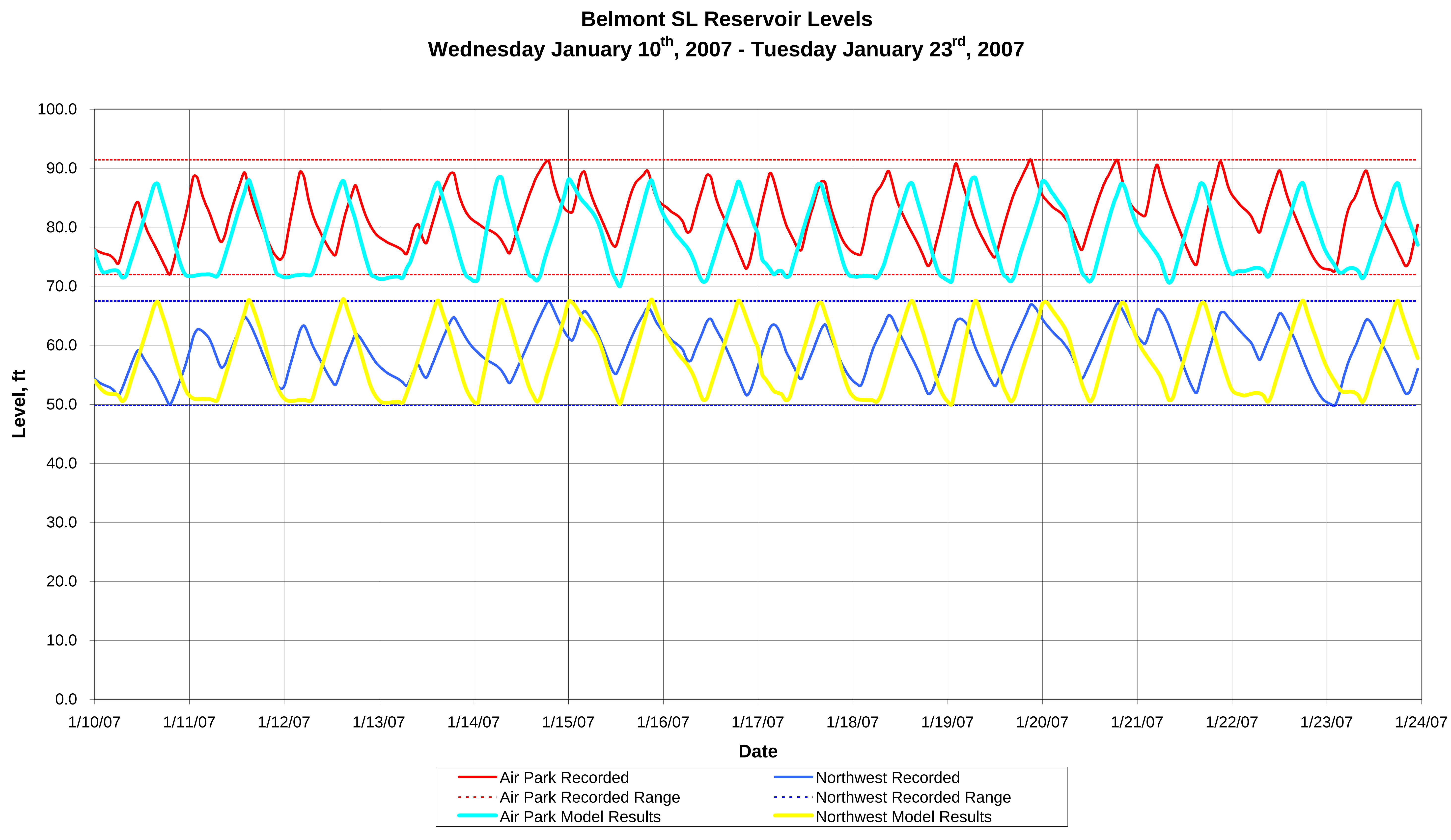
Conclusion

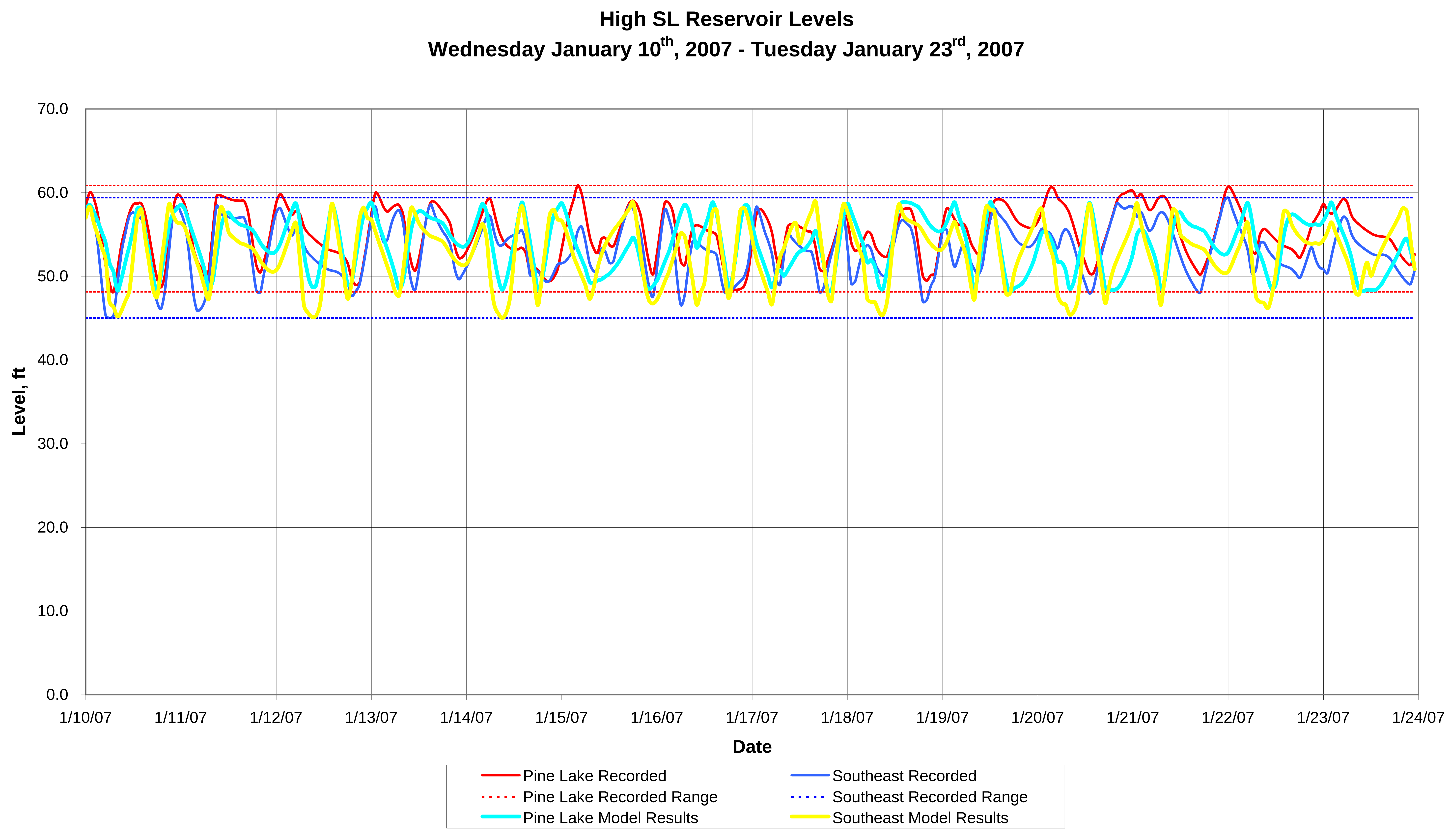
The results presented in this memorandum indicate that the diurnal patterns and operational parameters used in the water age analyses are indicative of current system operations and yield reliable and similar distribution system behaviors. Furthermore, the ability to achieve relatively close results to those recorded in SCADA corroborates the hydraulic model updates and the work performed in the hydraulic calibration efforts.

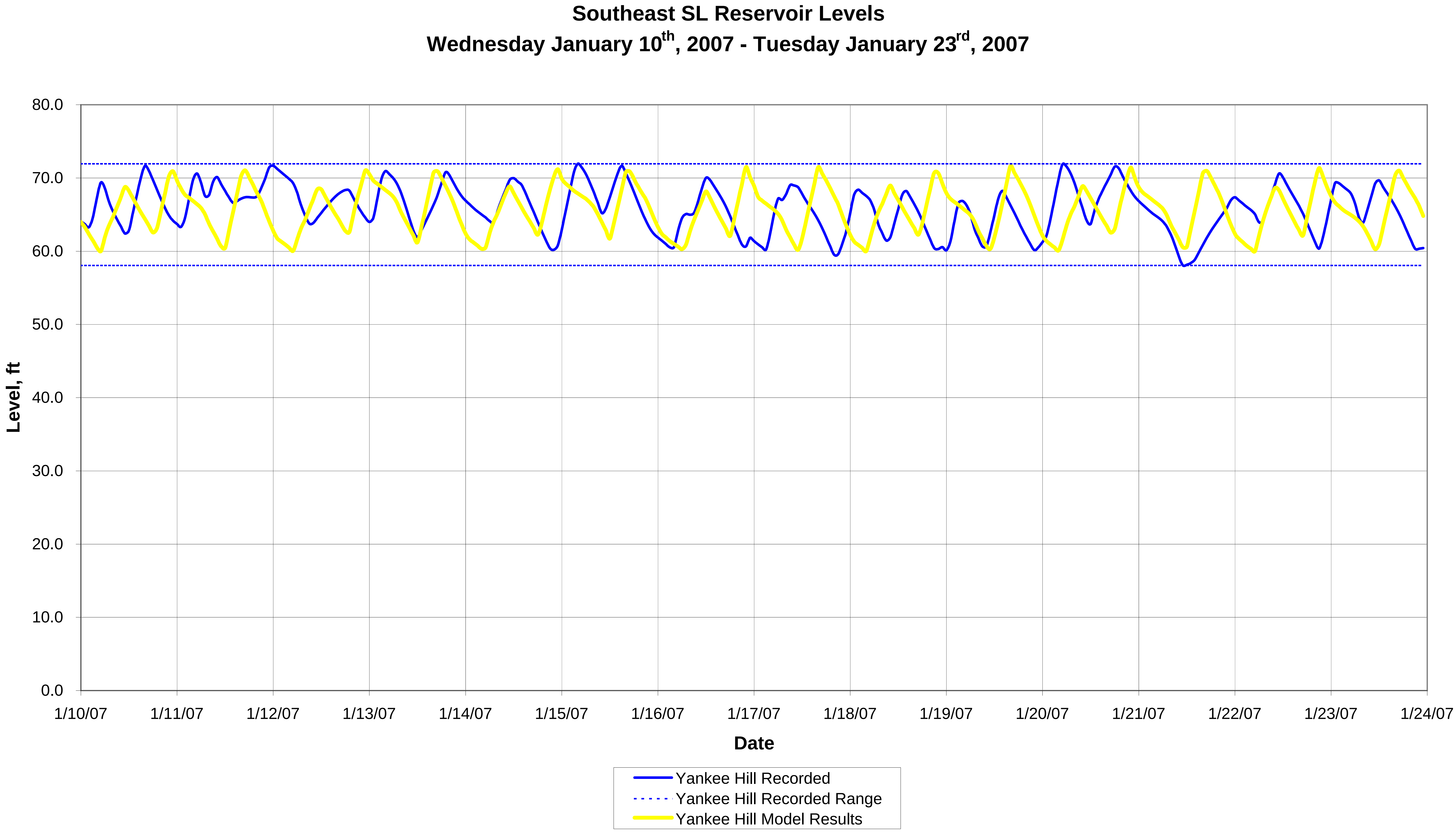

"A" Reservoir Levels
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
Level, ft
Reservoir 9 was out-of-service
for cleaning and inspection.
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
"A" Reservoir #5 Recorded
"A" Reservoir #6 Recorded
"A" Reservoir #9 Recorded *(Maintenance)
"A" Model Results
"A" Reservoir #5 Recorded Range
"A" Reservoir #6 Recorded Range
"A" Reservoir #9 Recorded Range *(Maintenance)



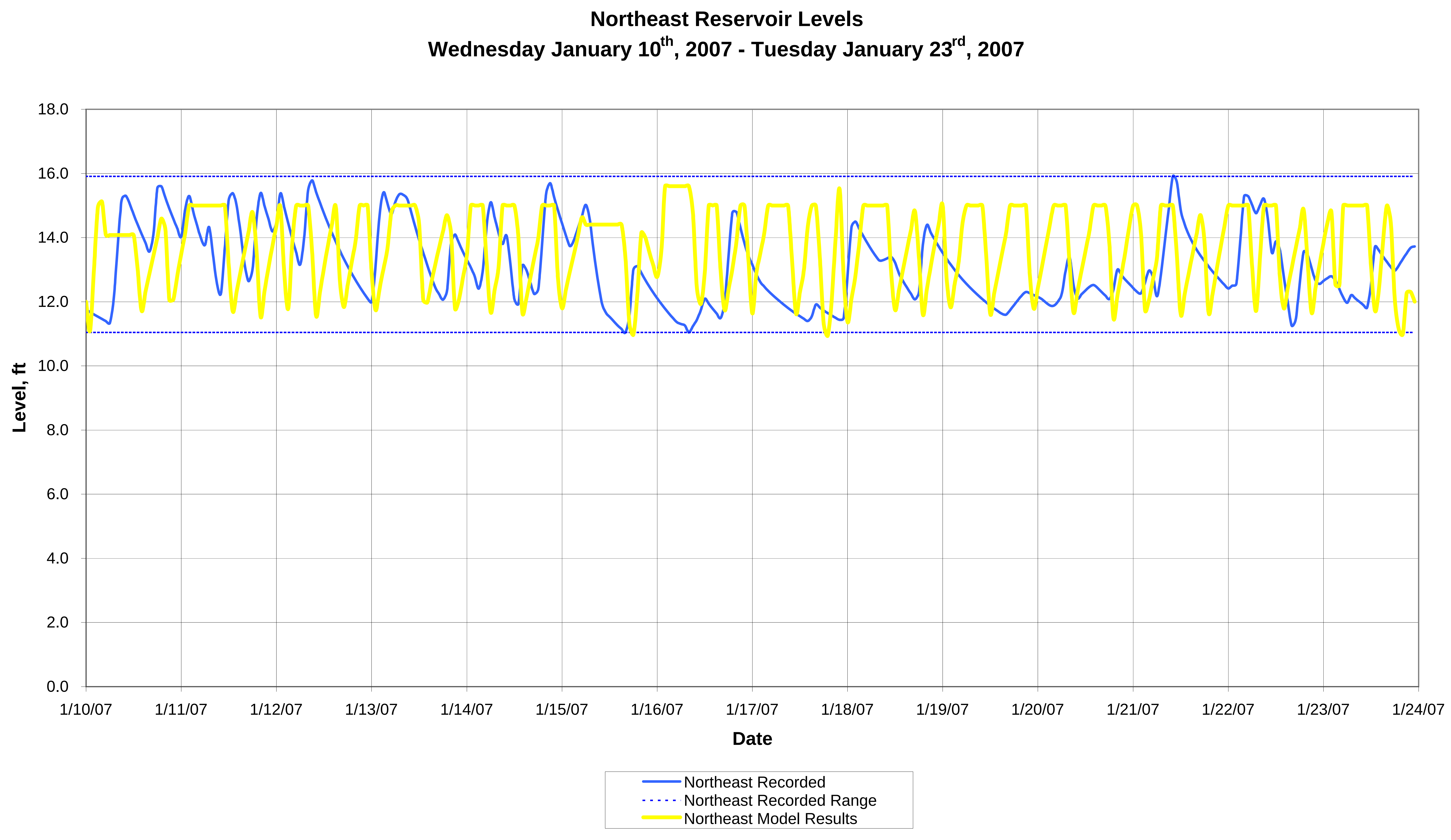
Low SL Reservoir Levels
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
Level, ft
60.0
50.0
40.0
30.0
20.0
10.0
0.0
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
Pioneer Recorded
Pioneer Recorded Range
Pioneer Model Results
Vine #1 Recorded
Vine Recorded Range
Vine Model Results


Belmont SL Reservoir Levels
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
Level, ft
100.0
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
Air Park Recorded
Air Park Recorded Range
Air Park Model Results
Northwest Recorded
Northwest Recorded Range
Northwest Model Results

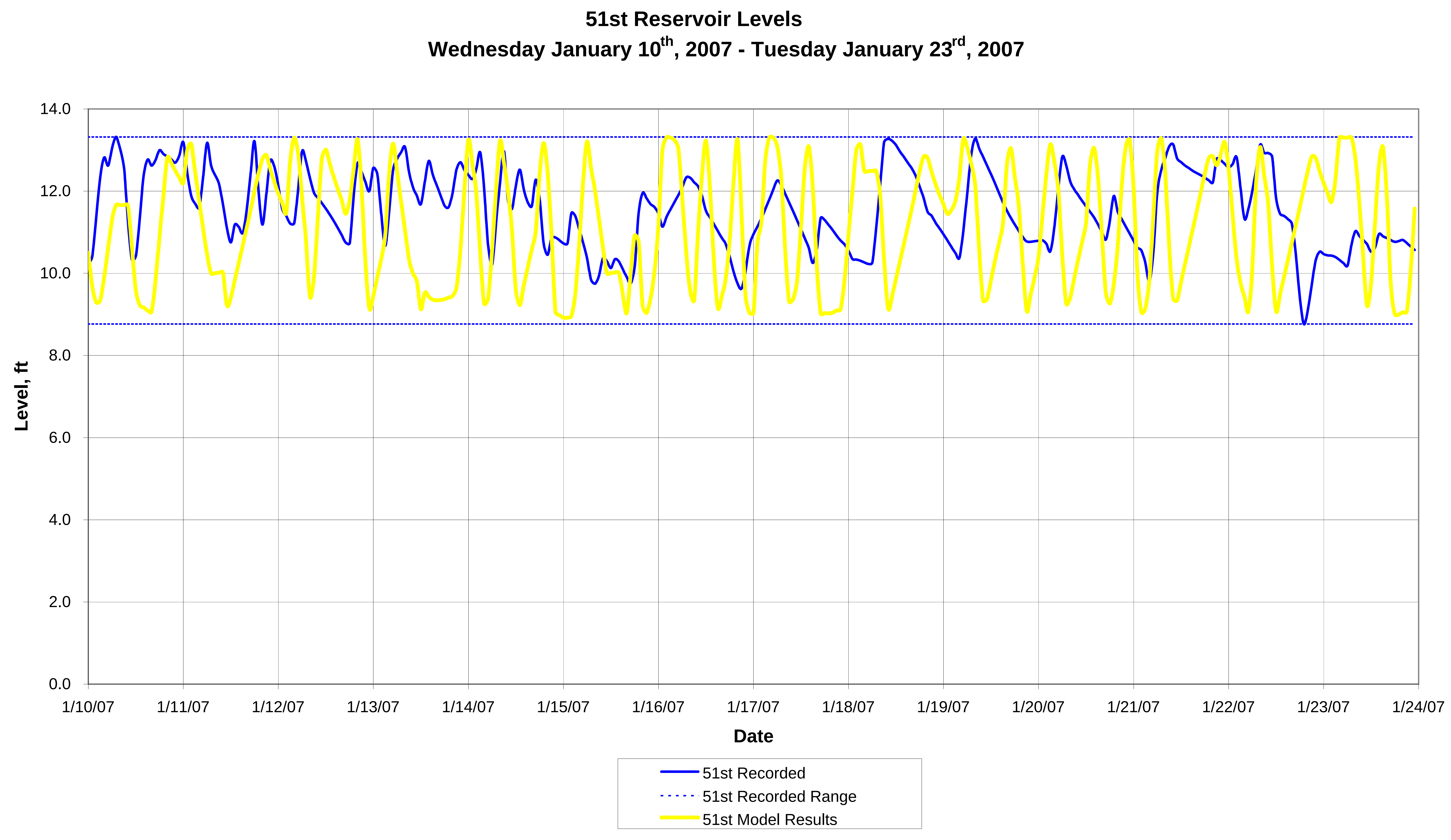

High SL Reservoir Levels
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
Level, ft
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
Pine Lake Recorded
Southeast Recorded
Pine Lake Recorded Range
Southeast Recorded Range
Pine Lake Model Results
Southeast Model Results

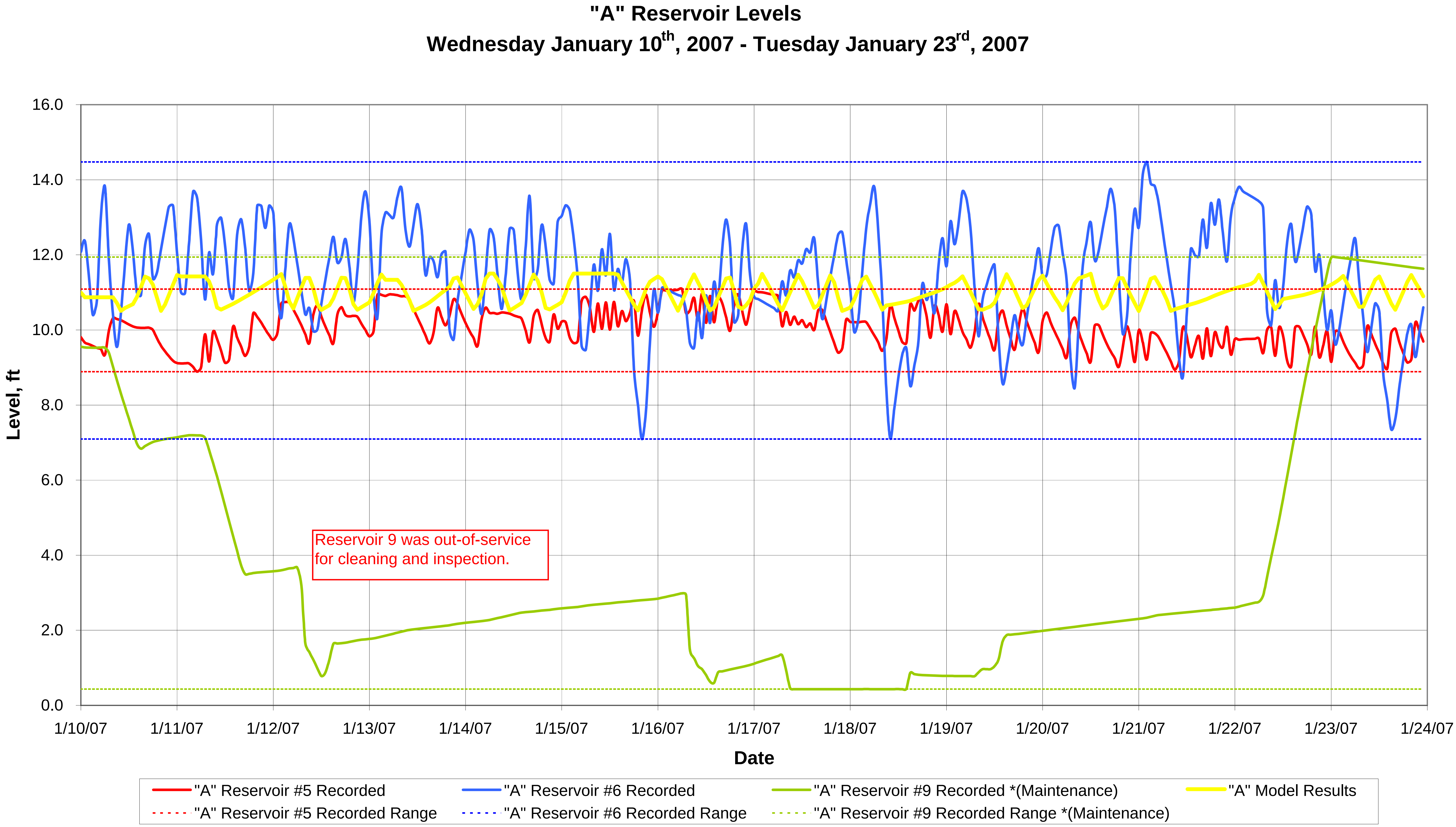


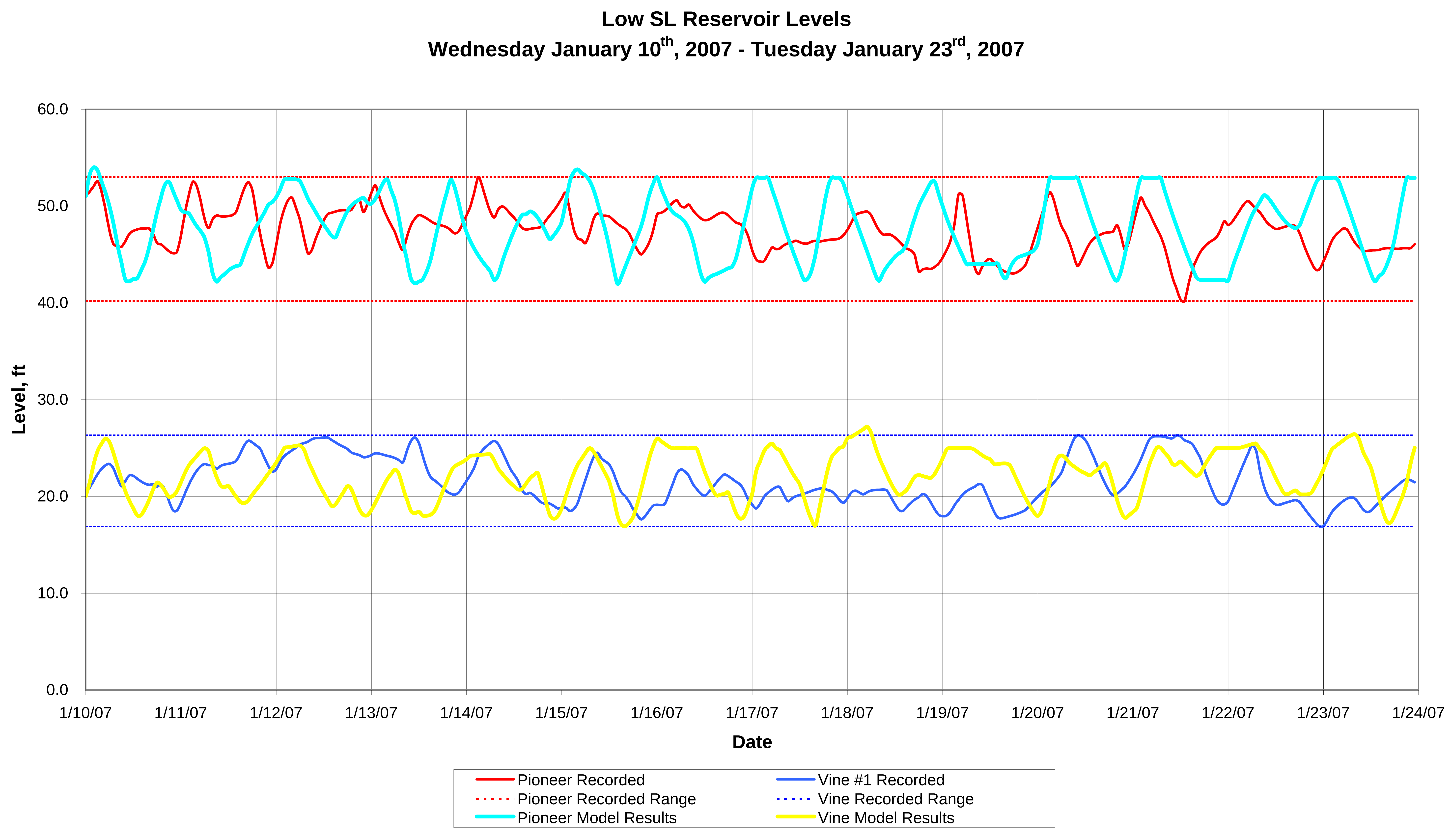
Southeast SL Reservoir Levels
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
Level, ft
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
Yankee Hill Recorded
Yankee Hill Recorded Range
Yankee Hill Model Results



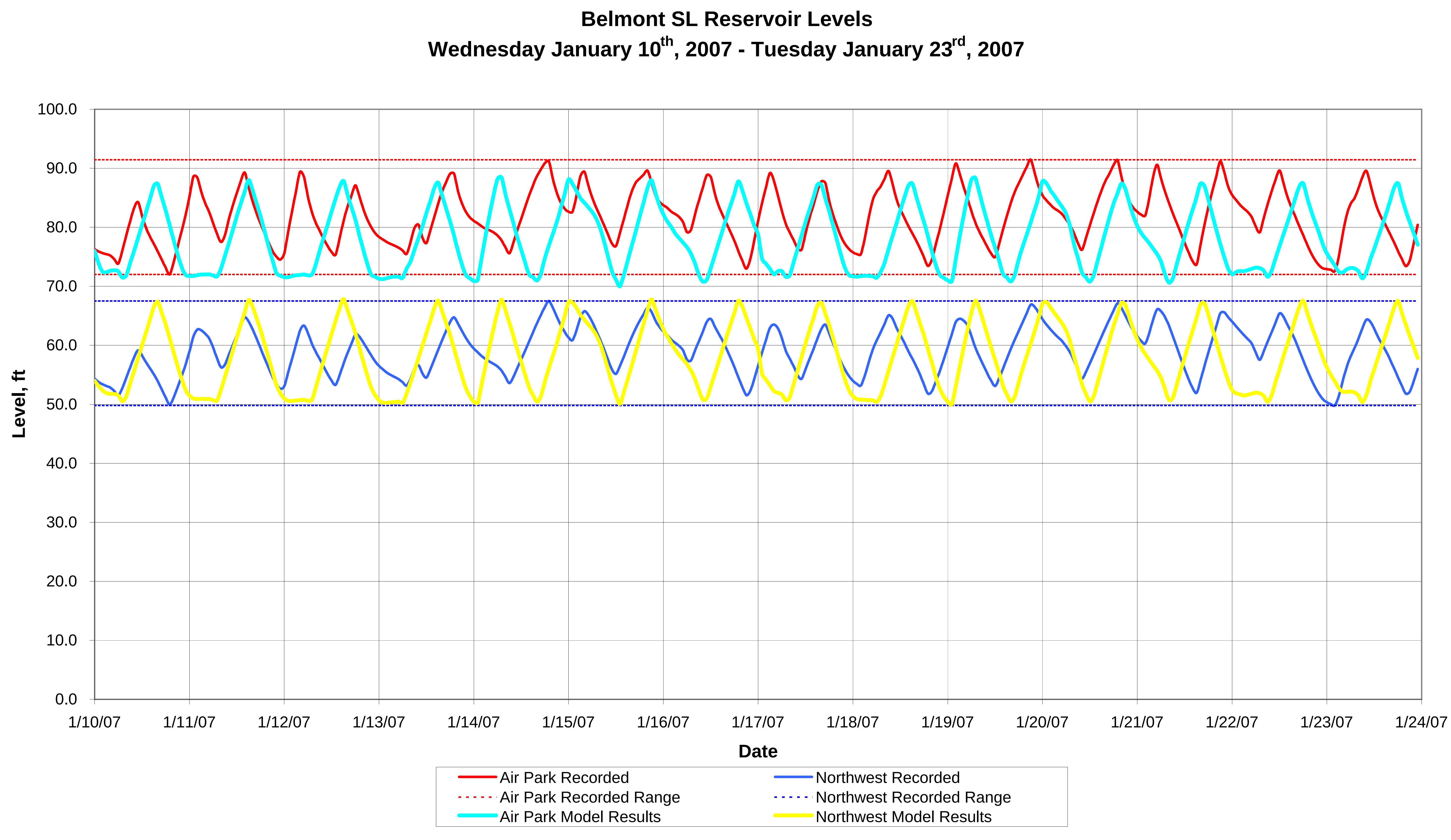
Northeast Reservoir Levels
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
Level, ft
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
Northeast Recorded
Northeast Recorded Range
Northeast Model Results


51st Reservoir Levels
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
Level, ft
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
51st Recorded
51st Recorded Range
51st Model Results

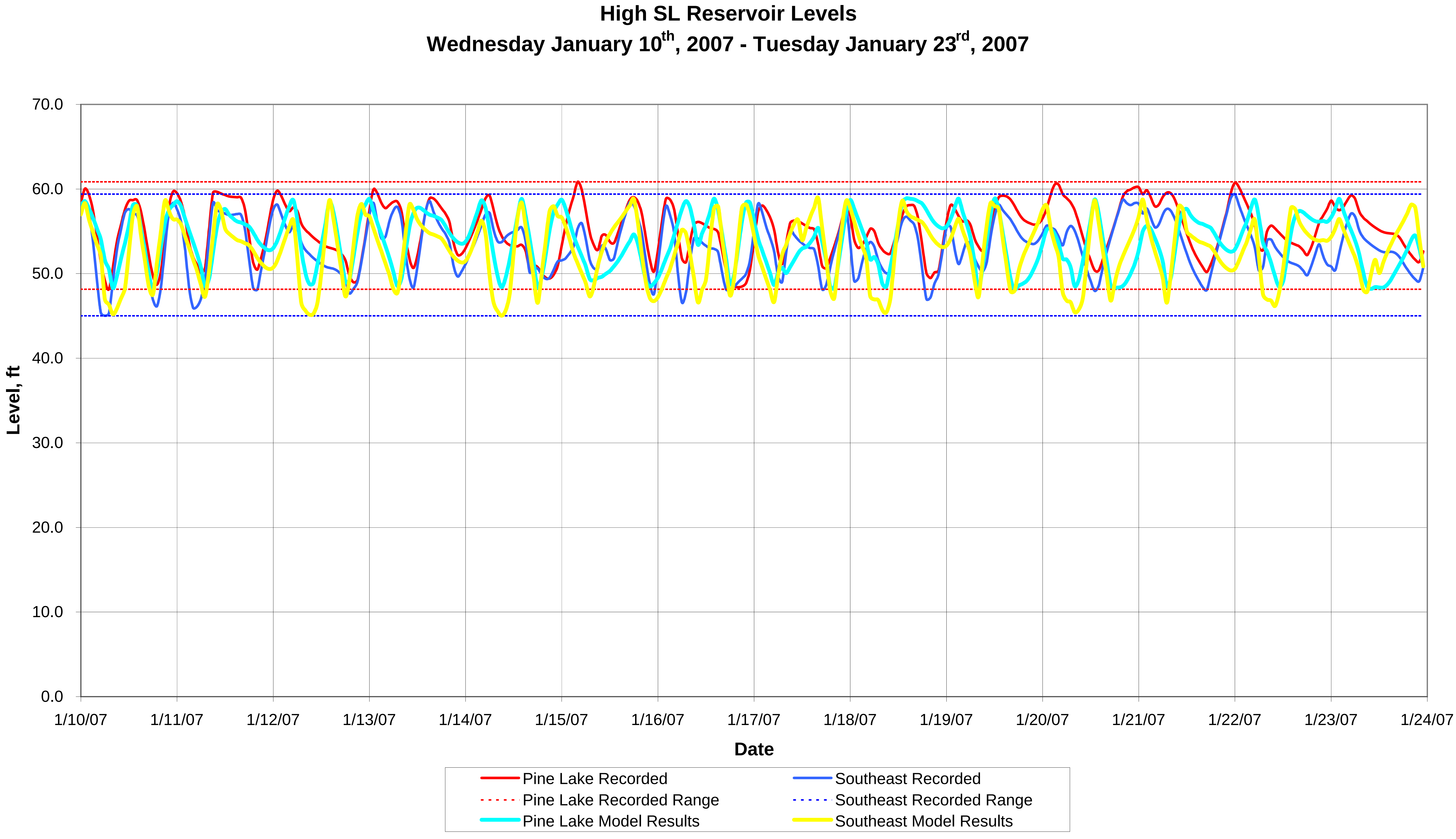

"A" Reservoir Levels
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
Level, ft
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
Reservoir 9 was out-of-service
for cleaning and inspection.
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
"A" Reservoir #5 Recorded
"A" Reservoir #6 Recorded
"A" Reservoir #9 Recorded *(Maintenance)
"A" Model Results
"A" Reservoir #5 Recorded Range
"A" Reservoir #6 Recorded Range
"A" Reservoir #9 Recorded Range *(Maintenance)



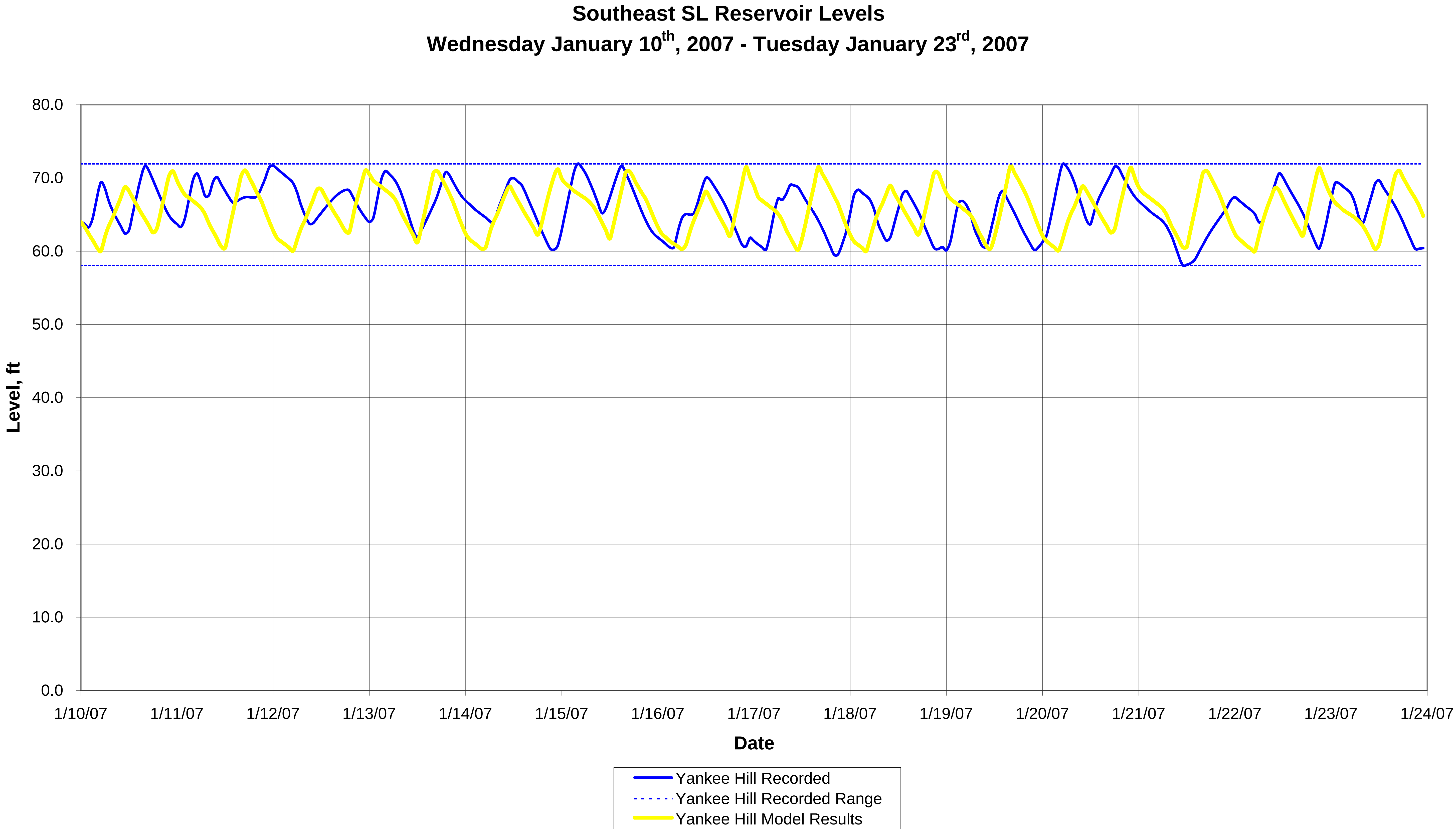
Low SL Reservoir Levels
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
60.0
50.0
40.0
30.0
20.0
10.0
0.0
Level, ft
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
Pioneer Recorded
Pioneer Recorded Range
Pioneer Model Results
Vine #1 Recorded
Vine Recorded Range
Vine Model Results



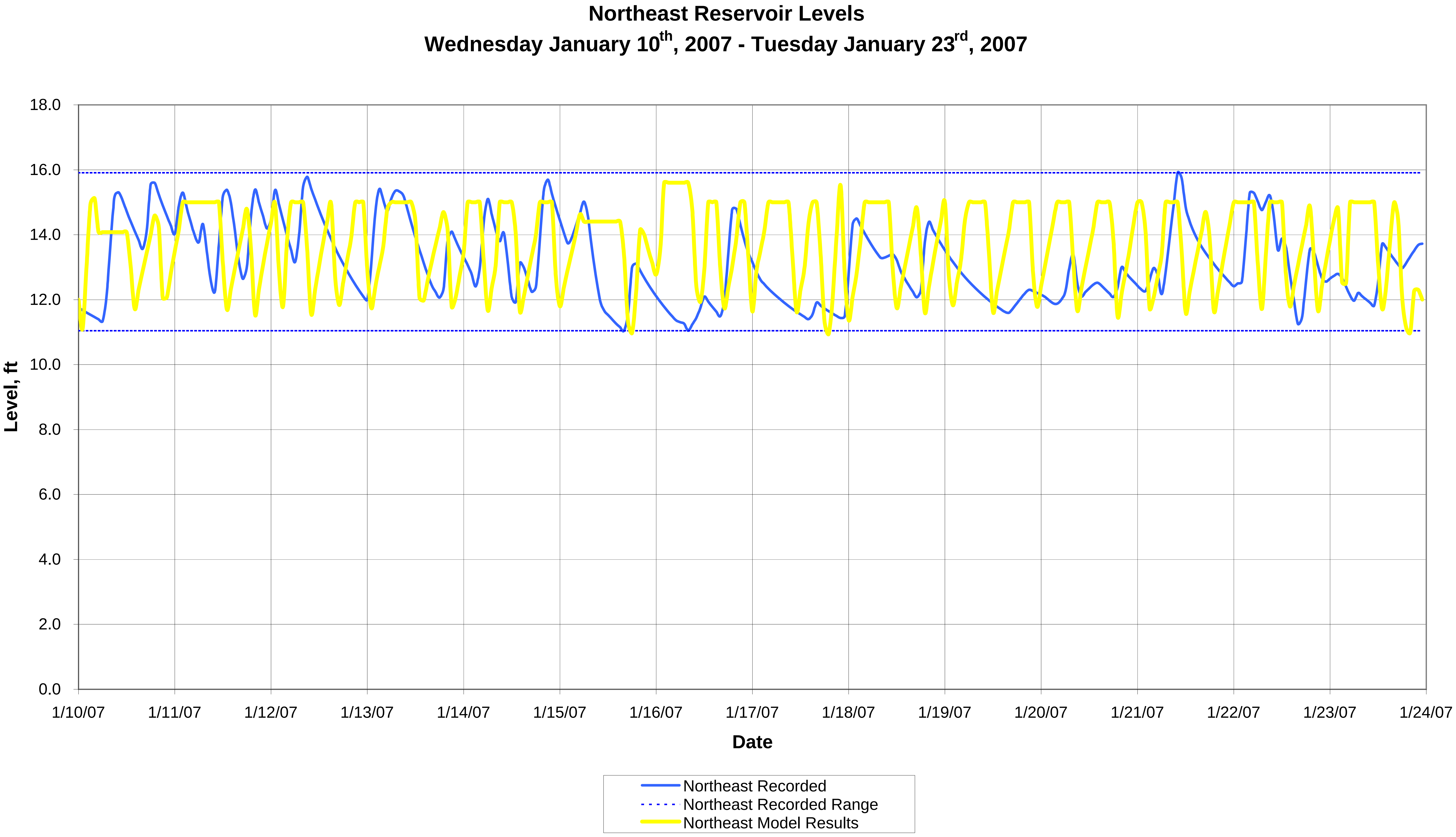
Belmont SL Reservoir Levels
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
Level, ft
100.0
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
Air Park Recorded
Air Park Recorded Range
Air Park Model Results
Northwest Recorded
Northwest Recorded Range
Northwest Model Results

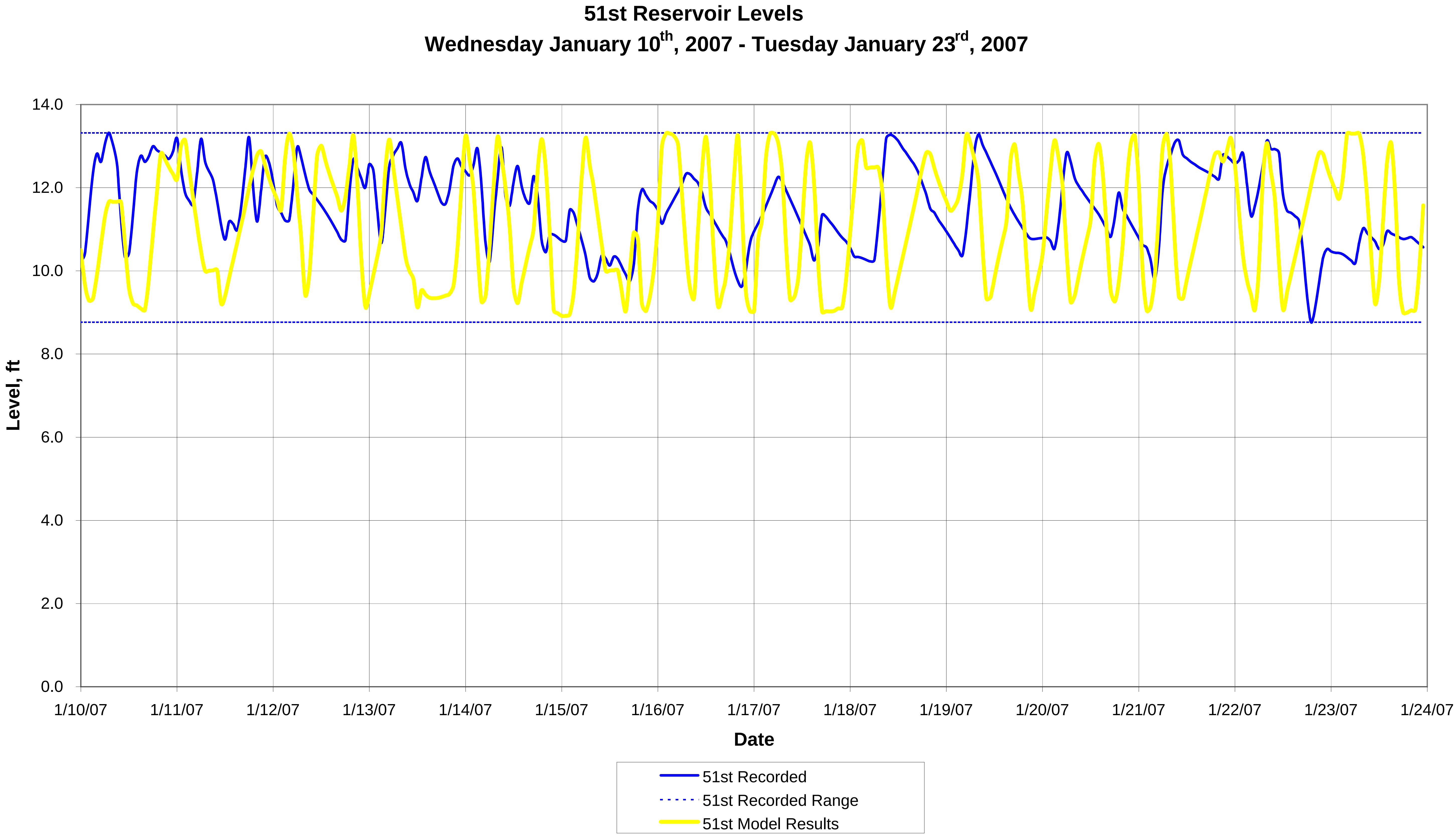


High SL Reservoir Levels
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
Level, ft
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
Pine Lake Recorded
Southeast Recorded
Pine Lake Recorded Range
Southeast Recorded Range
Pine Lake Model Results
Southeast Model Results

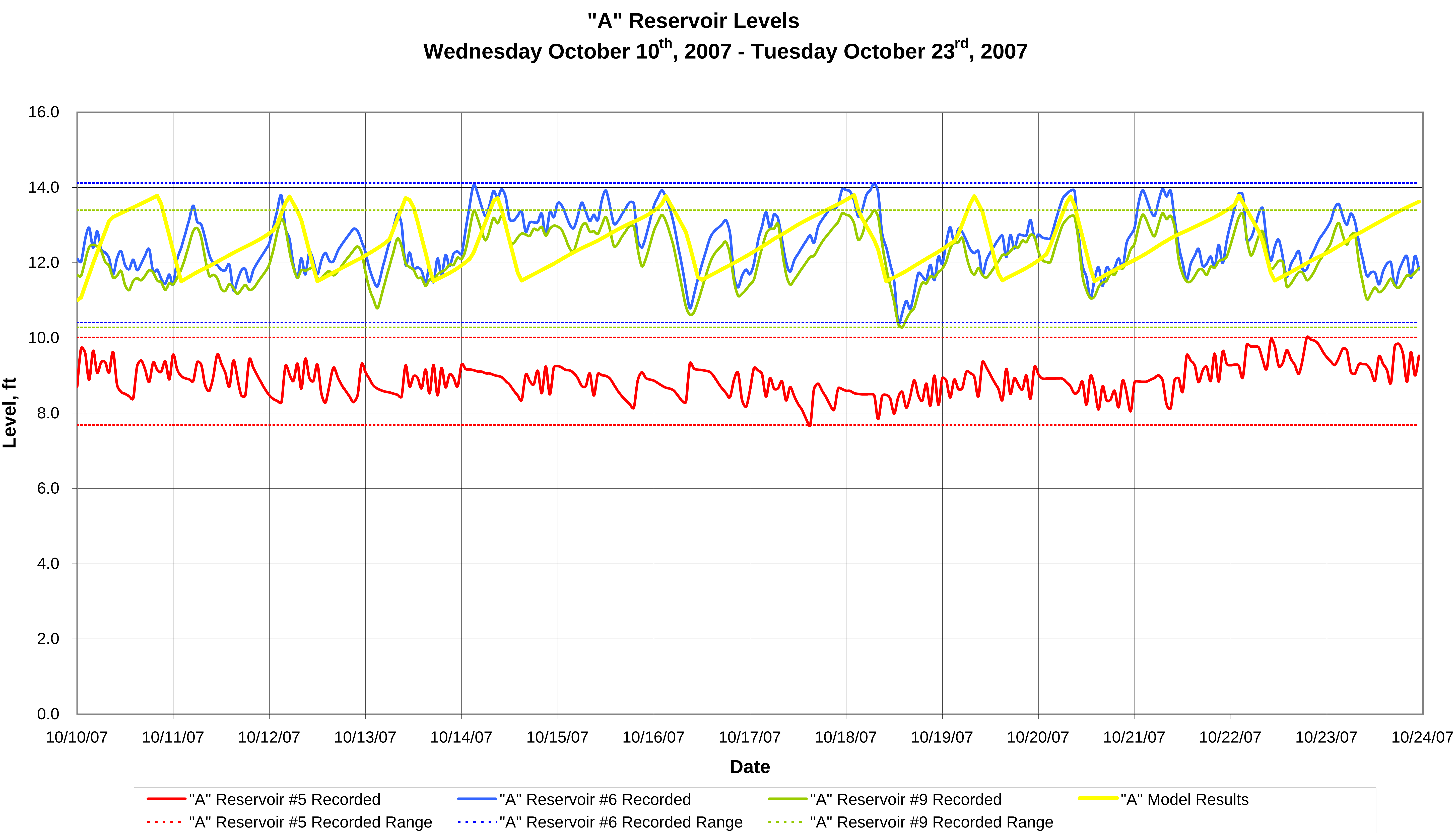


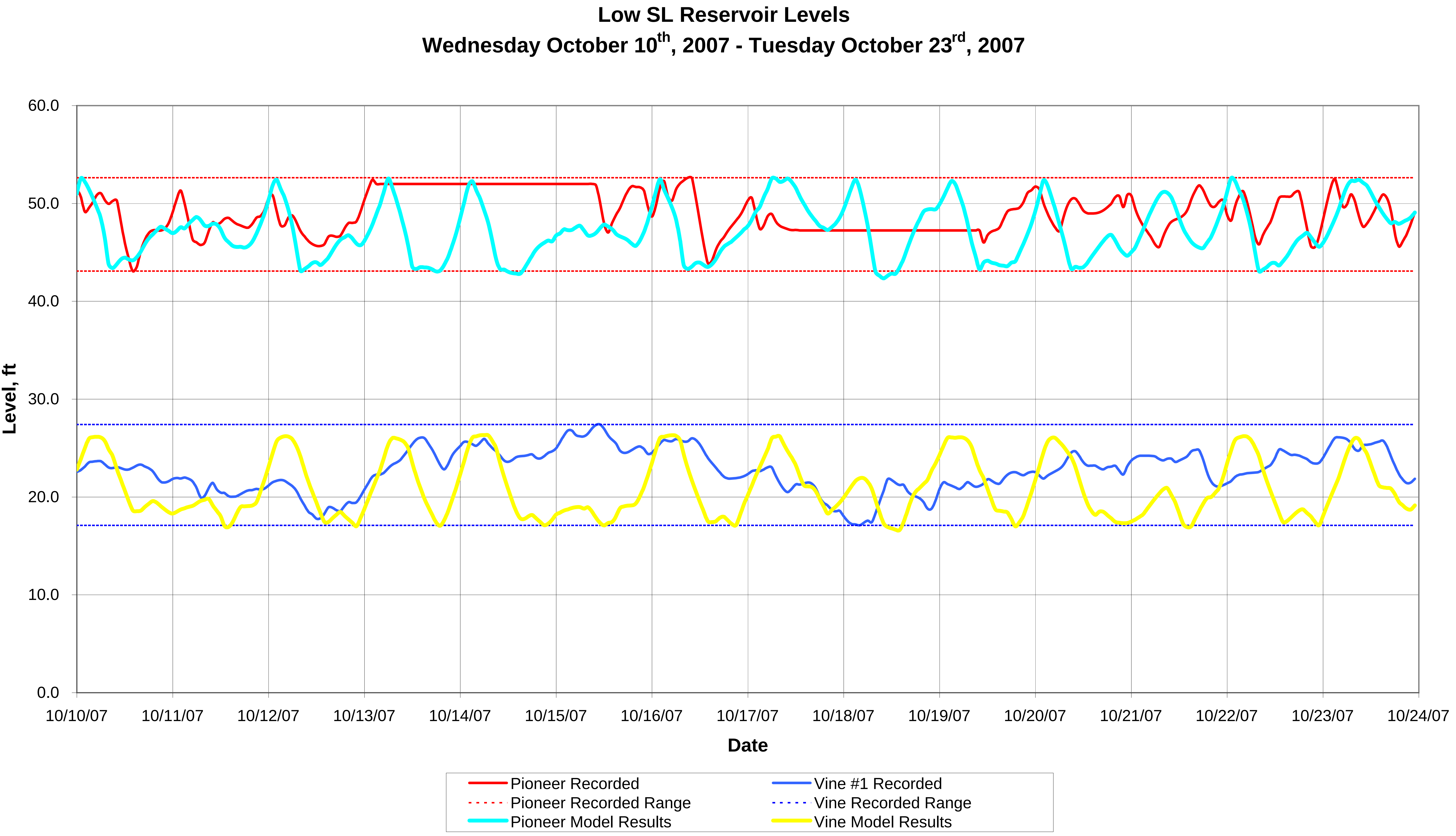
Southeast SL Reservoir Levels
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
Level, ft
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
Yankee Hill Recorded
Yankee Hill Recorded Range
Yankee Hill Model Results


Northeast Reservoir Levels
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
Level, ft
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
Northeast Recorded
Northeast Recorded Range
Northeast Model Results

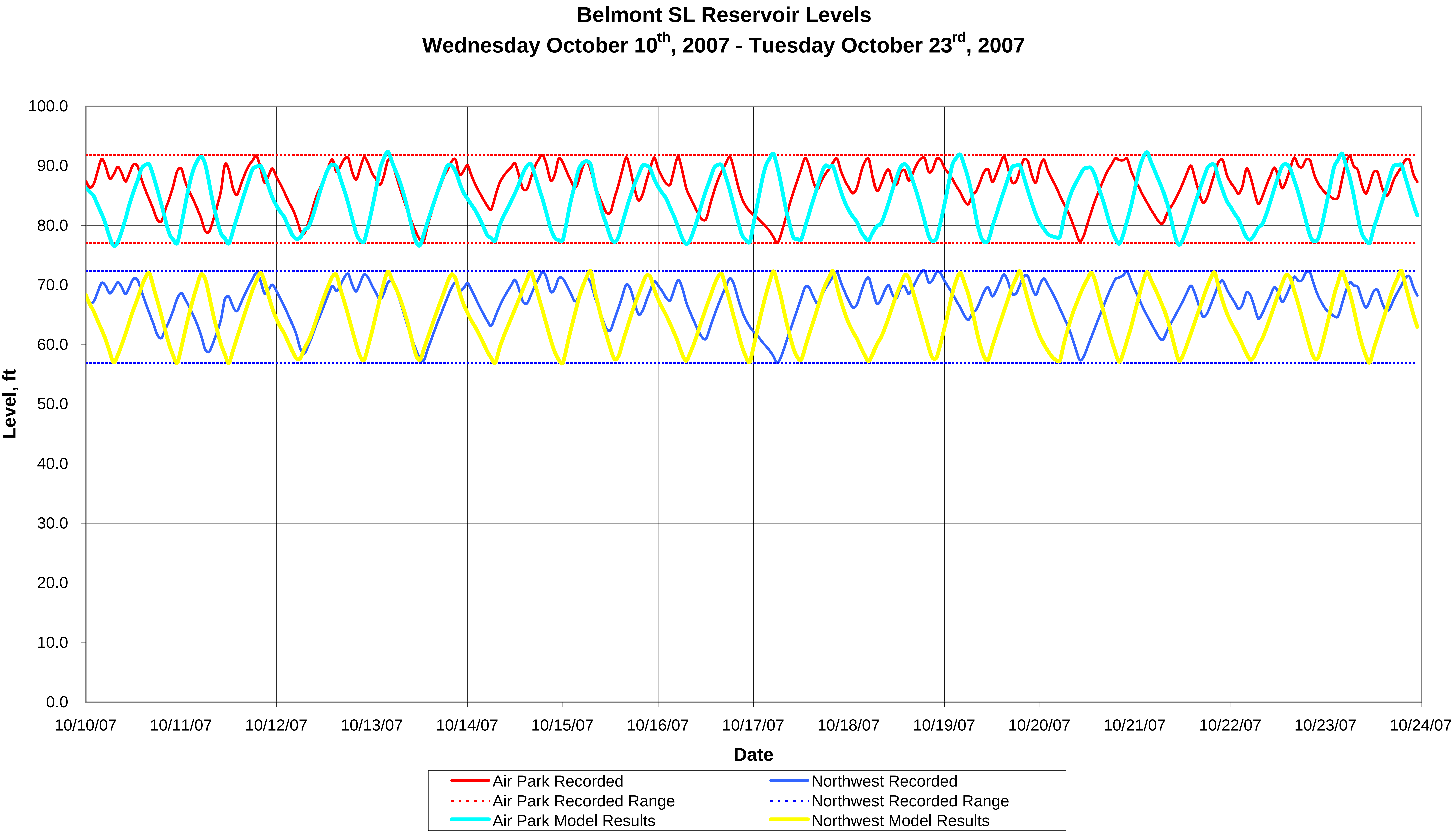


51st Reservoir Levels
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
Level, ft
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
51st Recorded
51st Recorded Range
51st Model Results

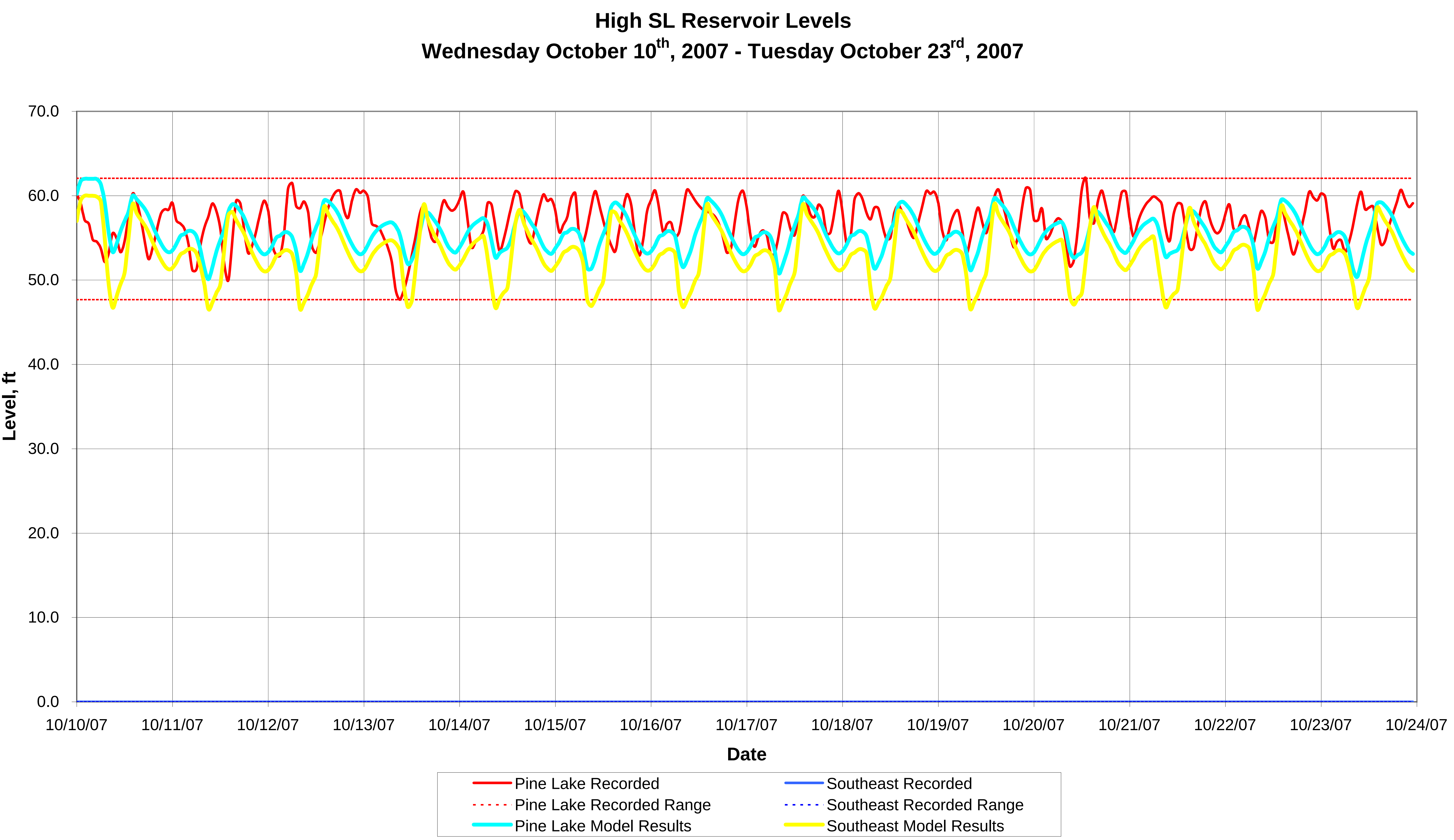

"A" Reservoir Levels
Wednesday October 10th, 2007 - Tuesday October 23rd, 2007
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
Level, ft
10/10/07
10/11/07
10/12/07
10/13/07
10/14/07
10/15/07
10/16/07
10/17/07
10/18/07
10/19/07
10/20/07
10/21/07
10/22/07
10/23/07
10/24/07
Date
"A" Reservoir #5 Recorded
"A" Reservoir #6 Recorded
"A" Reservoir #9 Recorded
"A" Model Results
"A" Reservoir #5 Recorded Range
"A" Reservoir #6 Recorded Range
"A" Reservoir #9 Recorded Range



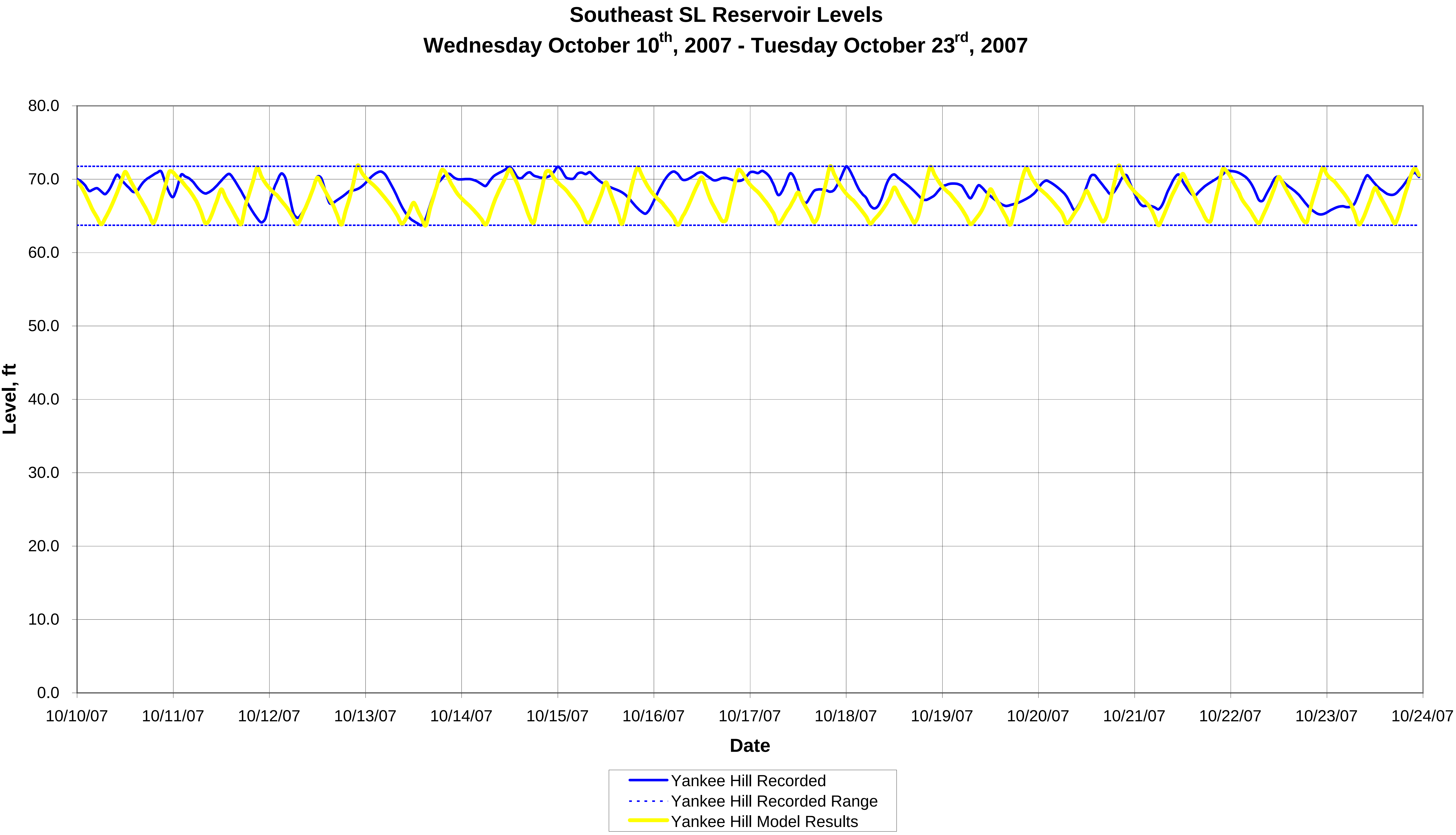
Low SL Reservoir Levels
Wednesday October 10th, 2007 - Tuesday October 23rd, 2007
Level, ft
60.0
50.0
40.0
30.0
20.0
10.0
0.0
10/10/07
10/11/07
10/12/07
10/13/07
10/14/07
10/15/07
10/16/07
10/17/07
10/18/07
10/19/07
10/20/07
10/21/07
10/22/07
10/23/07
10/24/07
Date
Pioneer Recorded
Pioneer Recorded Range
Pioneer Model Results
Vine #1 Recorded
Vine Recorded Range
Vine Model Results



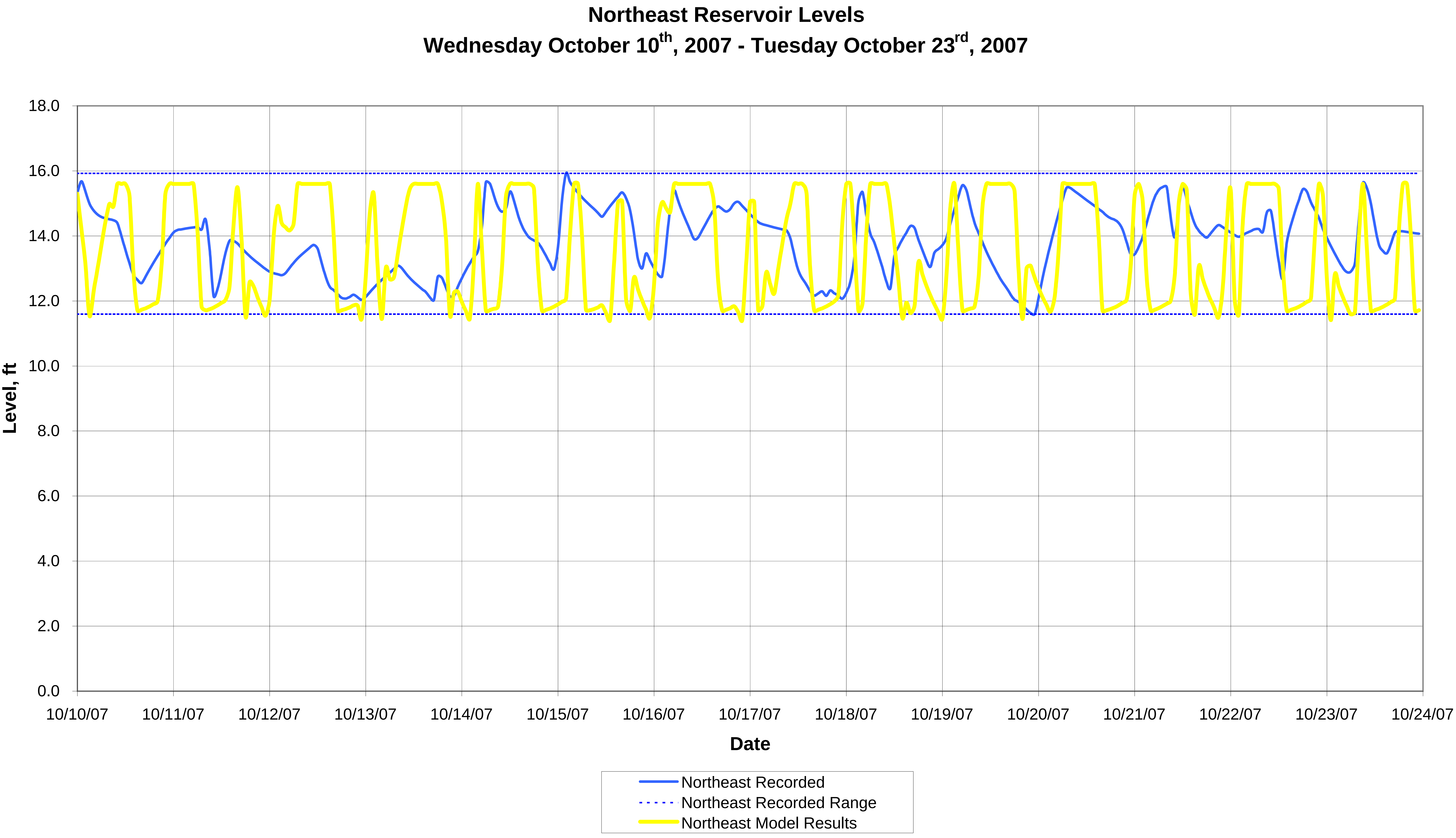
Belmont SL Reservoir Levels
Wednesday October 10th, 2007 - Tuesday October 23rd, 2007
100.0
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
Level, ft
10/10/07
10/11/07
10/12/07
10/13/07
10/14/07
10/15/07
10/16/07
10/17/07
10/18/07
10/19/07
10/20/07
10/21/07
10/22/07
10/23/07
10/24/07
Date
Air Park Recorded
Air Park Recorded Range
Air Park Model Results
Northwest Recorded
Northwest Recorded Range
Northwest Model Results



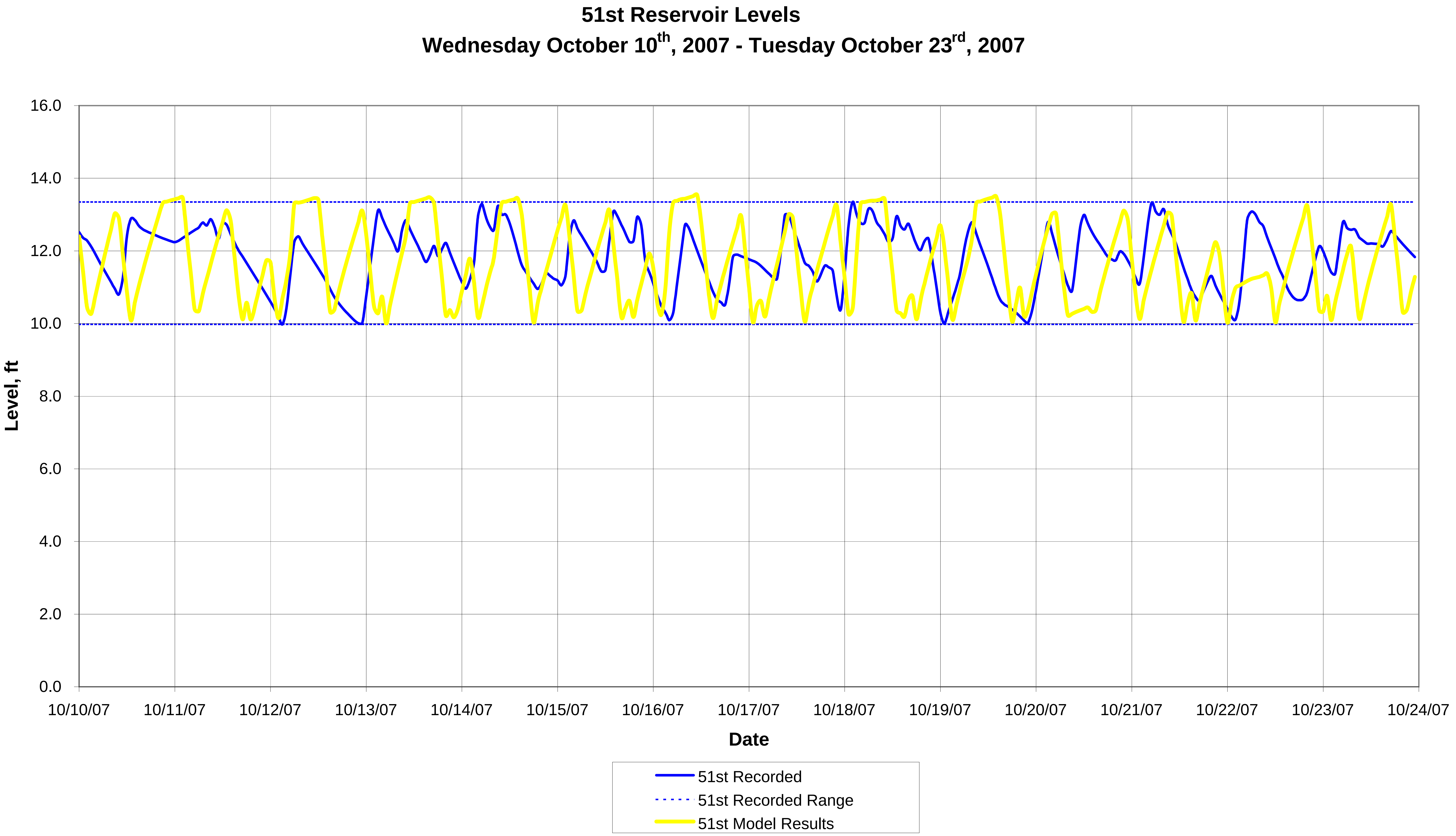
High SL Reservoir Levels
Wednesday October 10th, 2007 - Tuesday October 23rd, 2007
Level, ft
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
10/10/07
10/11/07
10/12/07
10/13/07
10/14/07
10/15/07
10/16/07
10/17/07
10/18/07
10/19/07
10/20/07
10/21/07
10/22/07
10/23/07
10/24/07
Date
Pine Lake Recorded
Pine Lake Recorded Range
Pine Lake Model Results
Southeast Recorded
Southeast Recorded Range
Southeast Model Results



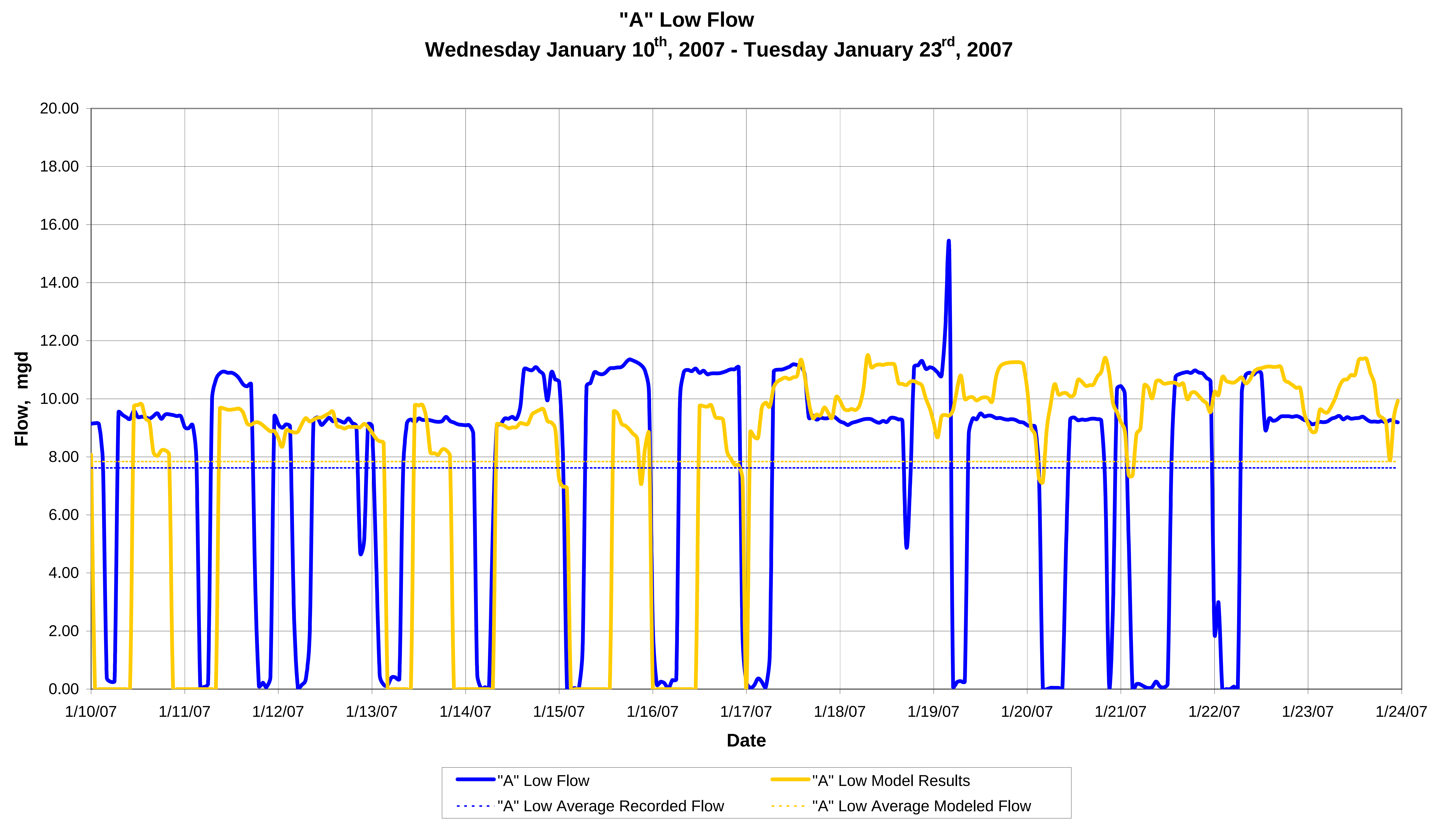
Southeast SL Reservoir Levels
Wednesday October 10th, 2007 - Tuesday October 23rd, 2007
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
Level, ft
10/10/07
10/11/07
10/12/07
10/13/07
10/14/07
10/15/07
10/16/07
10/17/07
10/18/07
10/19/07
10/20/07
10/21/07
10/22/07
10/23/07
10/24/07
Date
Yankee Hill Recorded
Yankee Hill Recorded Range
Yankee Hill Model Results



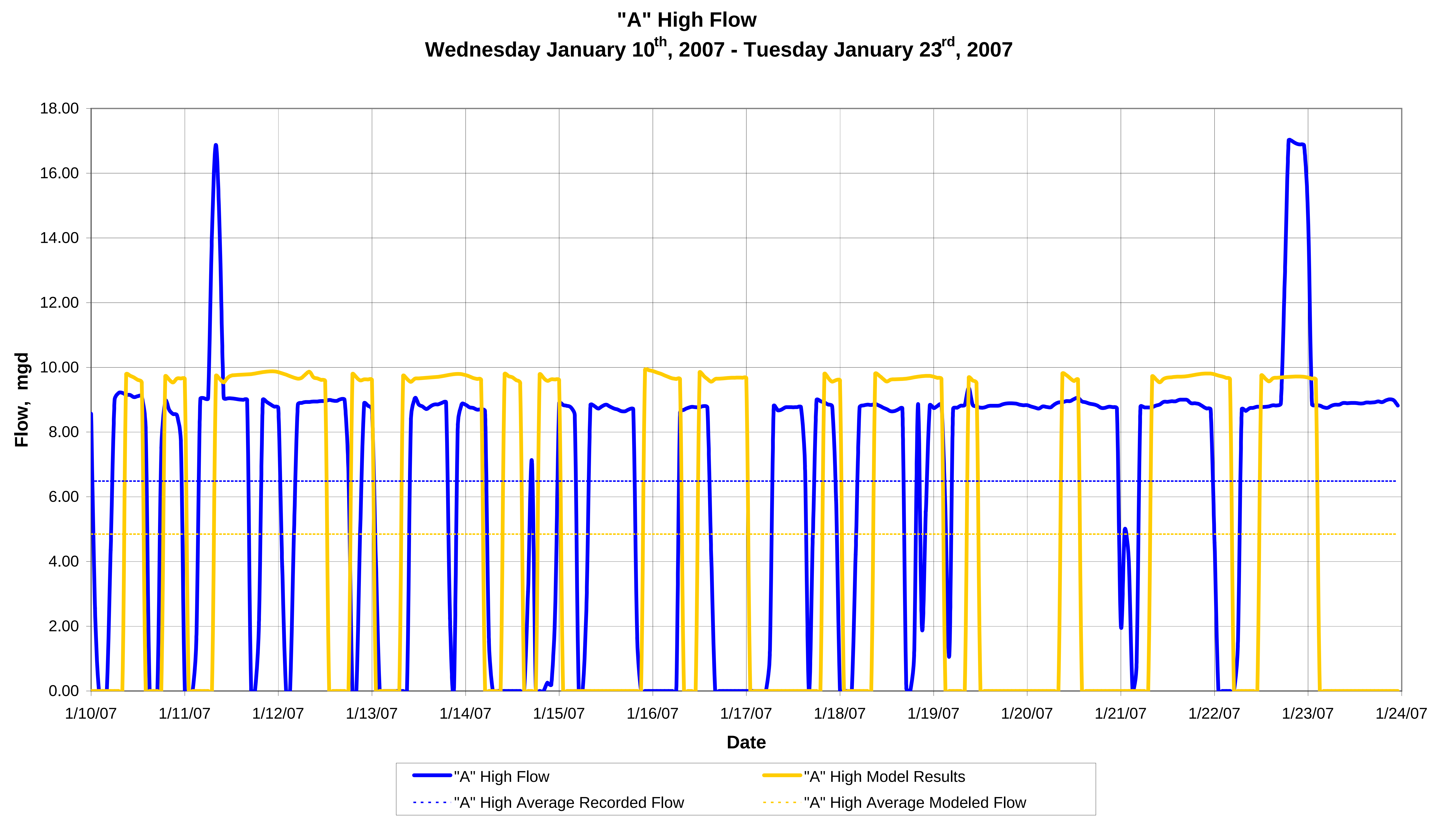
Northeast Reservoir Levels
Wednesday October 10th, 2007 - Tuesday October 23rd, 2007
Level, ft
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
10/10/07
10/11/07
10/12/07
10/13/07
10/14/07
10/15/07
10/16/07
10/17/07
10/18/07
10/19/07
10/20/07
10/21/07
10/22/07
10/23/07
10/24/07
Date
Northeast Recorded
Northeast Recorded Range
Northeast Model Results

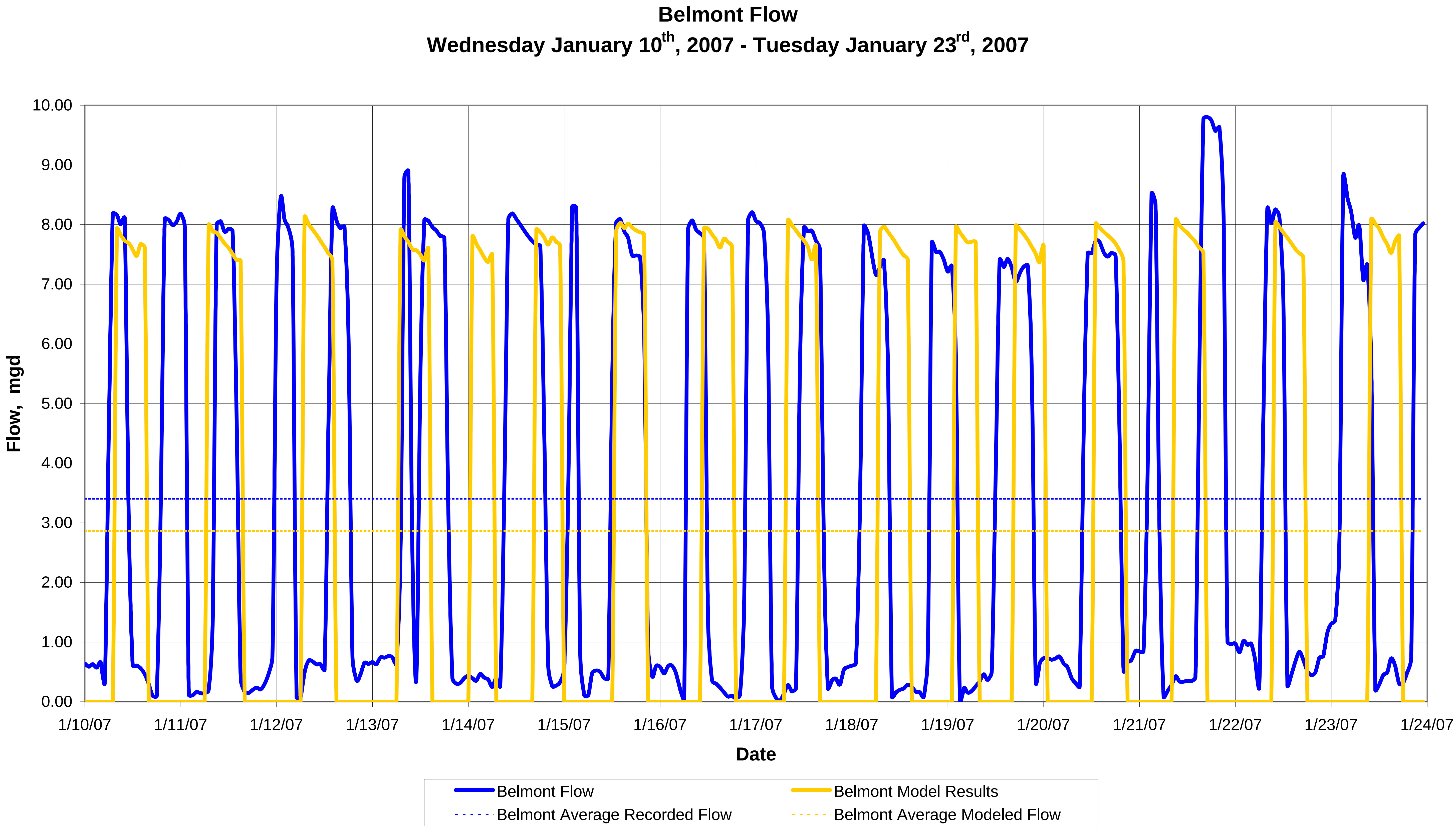


51st Reservoir Levels
Wednesday October 10th, 2007 - Tuesday October 23rd, 2007
Level, ft
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
10/10/07
10/11/07
10/12/07
10/13/07
10/14/07
10/15/07
10/16/07
10/17/07
10/18/07
10/19/07
10/20/07
10/21/07
10/22/07
10/23/07
10/24/07
Date
51st Recorded
51st Recorded Range
51st Model Results

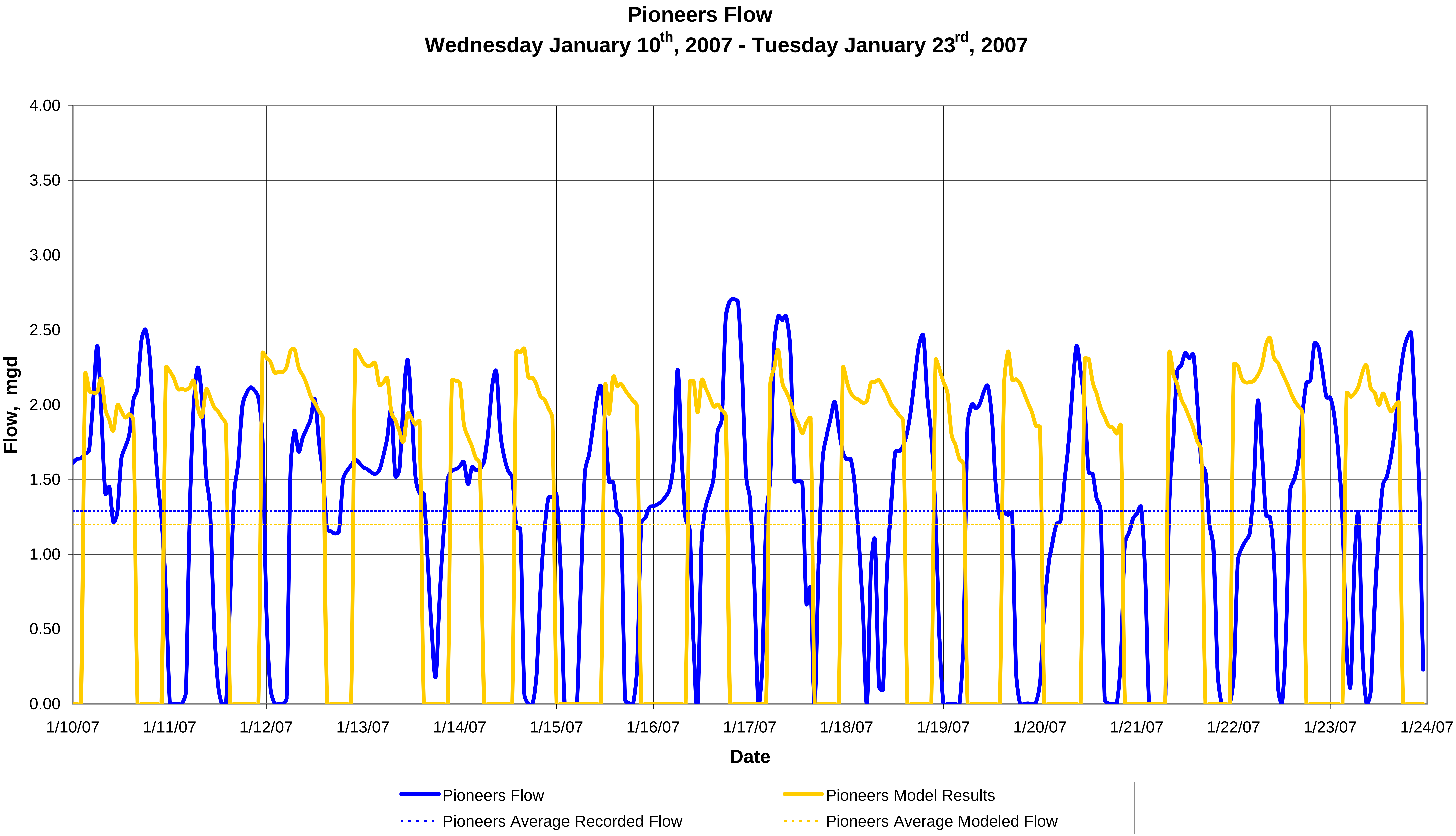


"A" Low Flow
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
Flow, mgd
20.00
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
"A" Low Flow
"A" Low Model Results
"A" Low Average Recorded Flow
"A" Low Average Modeled Flow

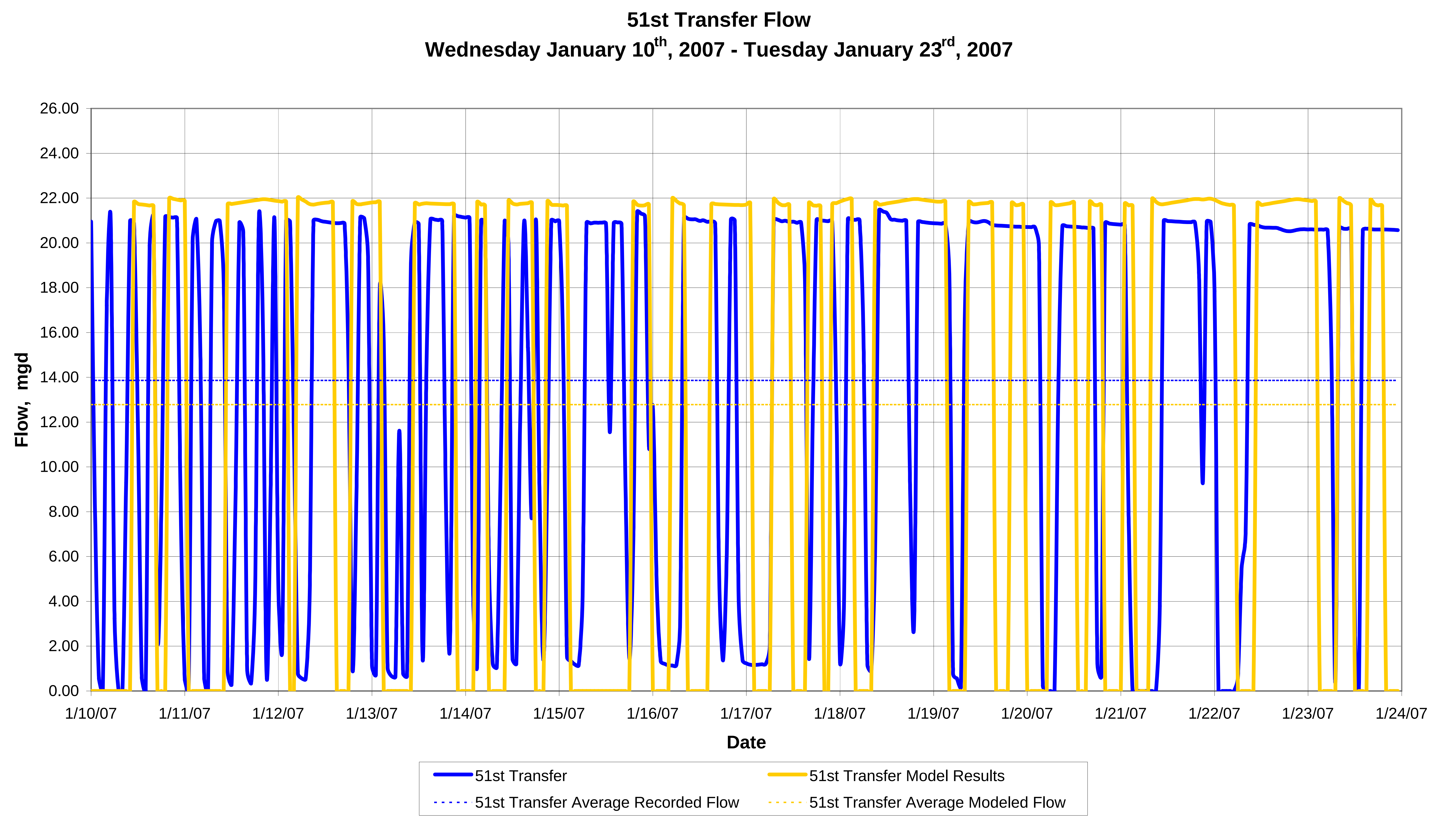

"A" High Flow
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
Flow, mgd
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
"A" High Flow
"A" High Model Results
"A" High Average Recorded Flow
"A" High Average Modeled Flow

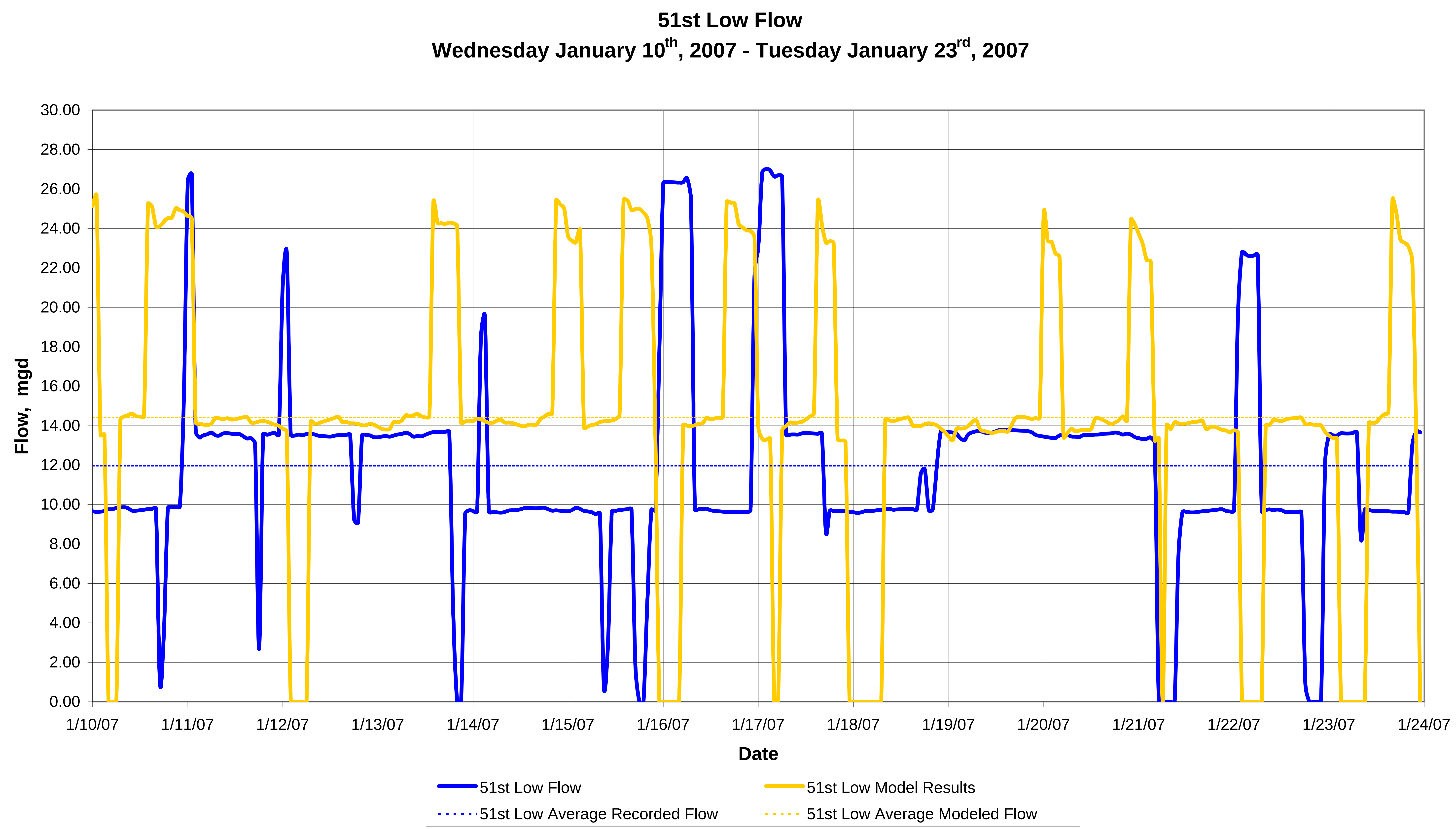


Belmont Flow
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
Flow, mgd
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
Belmont Flow
Belmont Model Results
Belmont Average Recorded Flow
Belmont Average Modeled Flow

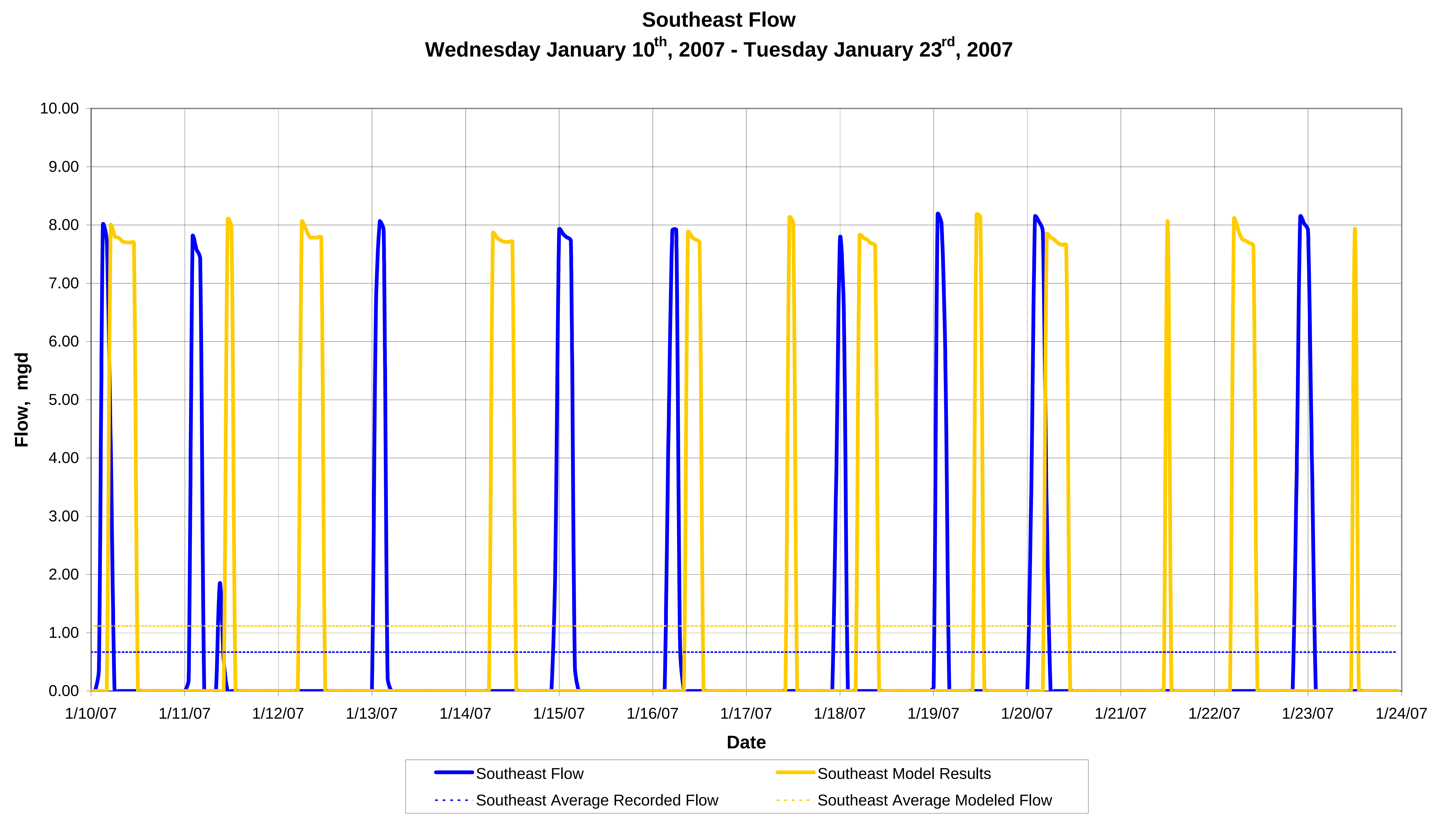


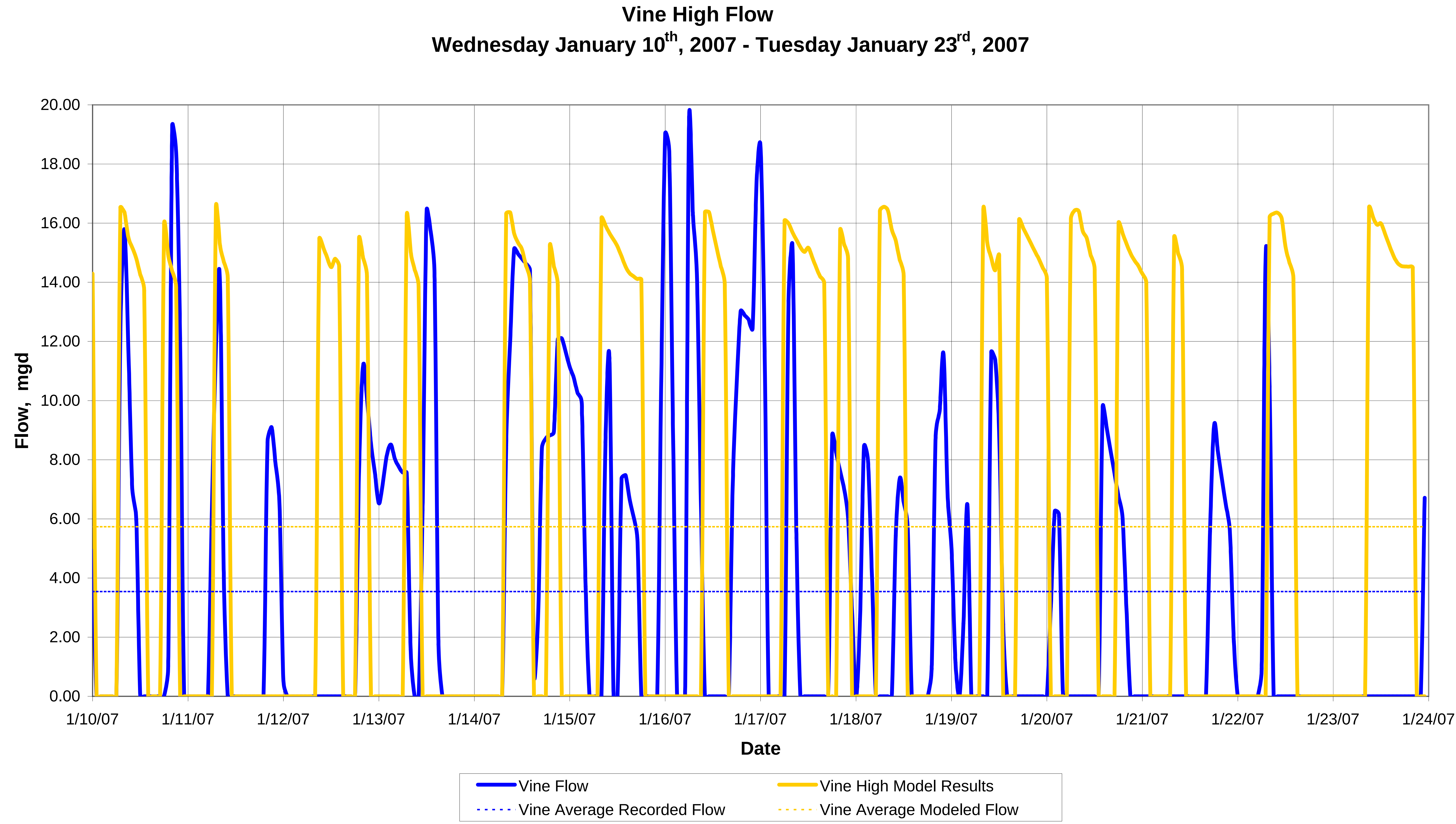
Pioneers Flow
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Flow, mgd
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
Pioneers Flow
Pioneers Model Results
Pioneers Average Recorded Flow
Pioneers Average Modeled Flow


51st Transfer Flow
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
Flow, mgd
26.00
24.00
22.00
20.00
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
51st Transfer
51st Transfer Model Results
51st Transfer Average Recorded Flow
51st Transfer Average Modeled Flow



51st Low Flow
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
Flow, mgd
30.00
28.00
26.00
24.00
22.00
20.00
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
51st Low Flow
51st Low Model Results
51st Low Average Recorded Flow
51st Low Average Modeled Flow

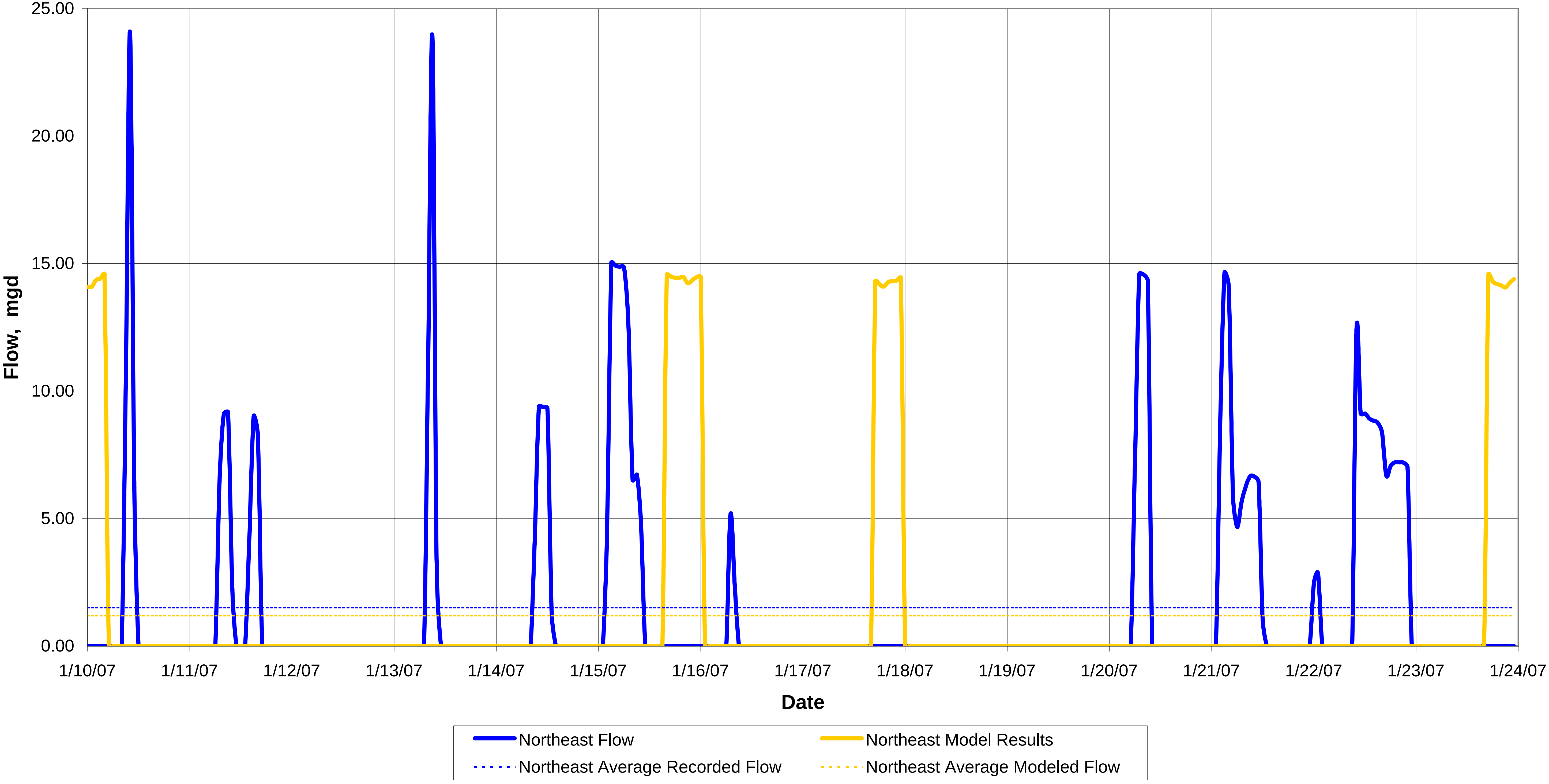

Southeast Flow
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
Flow, mgd
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
Southeast Flow
Southeast Model Results
Southeast Average Recorded Flow
Southeast Average Modeled Flow

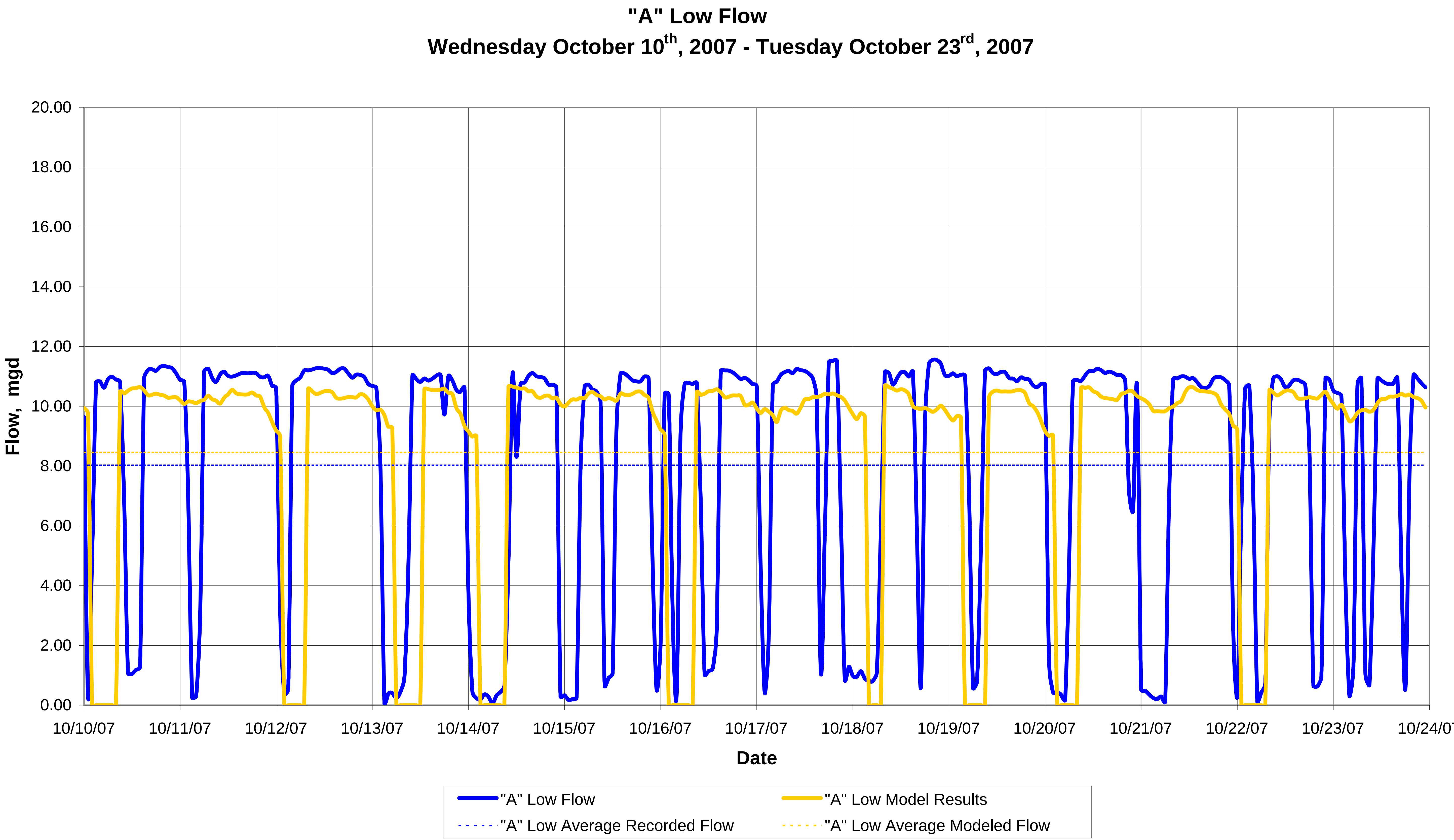


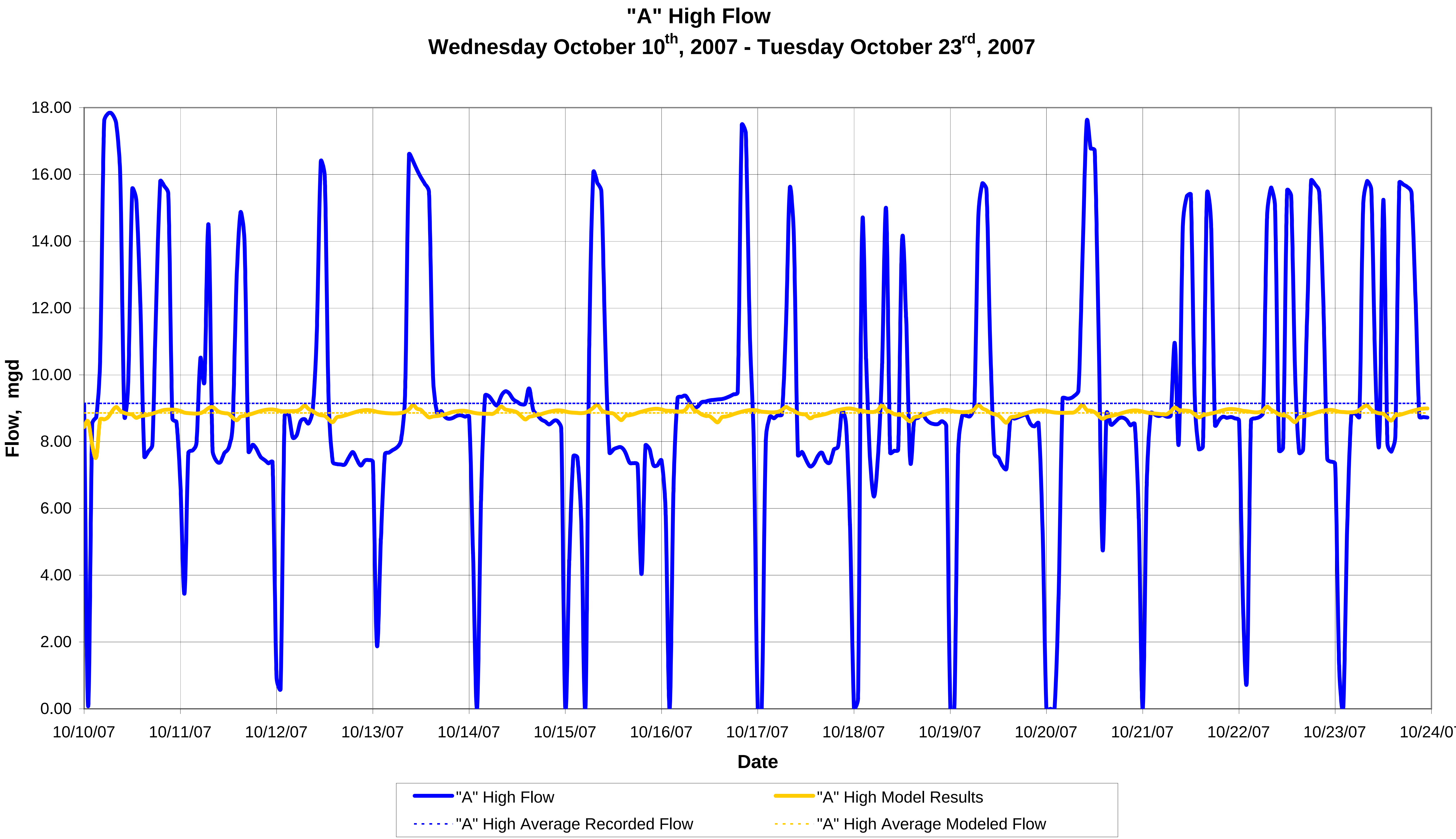
Vine High Flow
Wednesday January 10th, 2007 - Tuesday January 23rd, 2007
Flow, mgd
20.00
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
1/10/07
1/11/07
1/12/07
1/13/07
1/14/07
1/15/07
1/16/07
1/17/07
1/18/07
1/19/07
1/20/07
1/21/07
1/22/07
1/23/07
1/24/07
Date
Vine Flow
Vine High Model Results
Vine Average Recorded Flow
Vine Average Modeled Flow

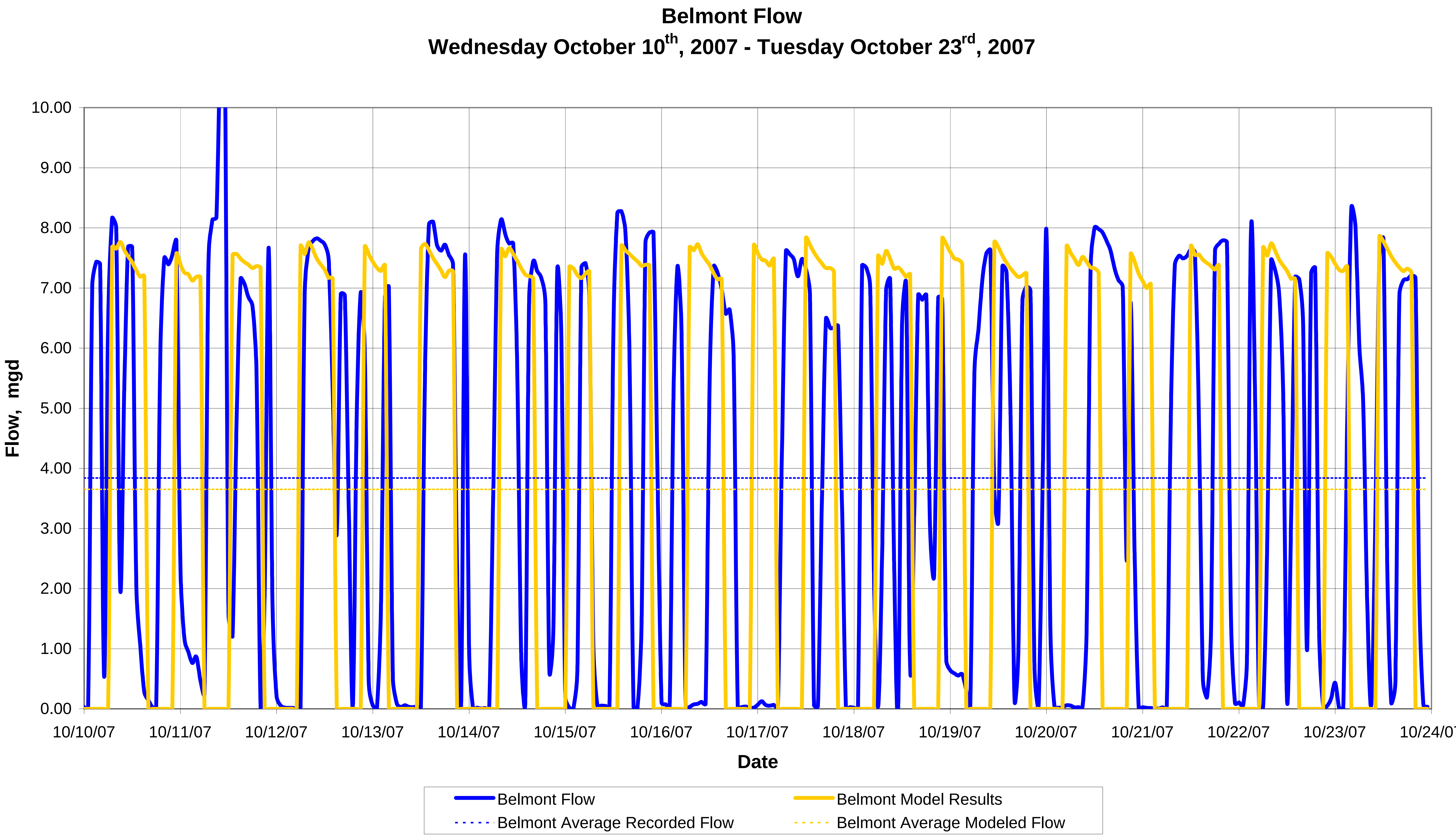
Northeast Flow

Wednesday January 10th, 2007 - Tuesday January 23rd, 2007



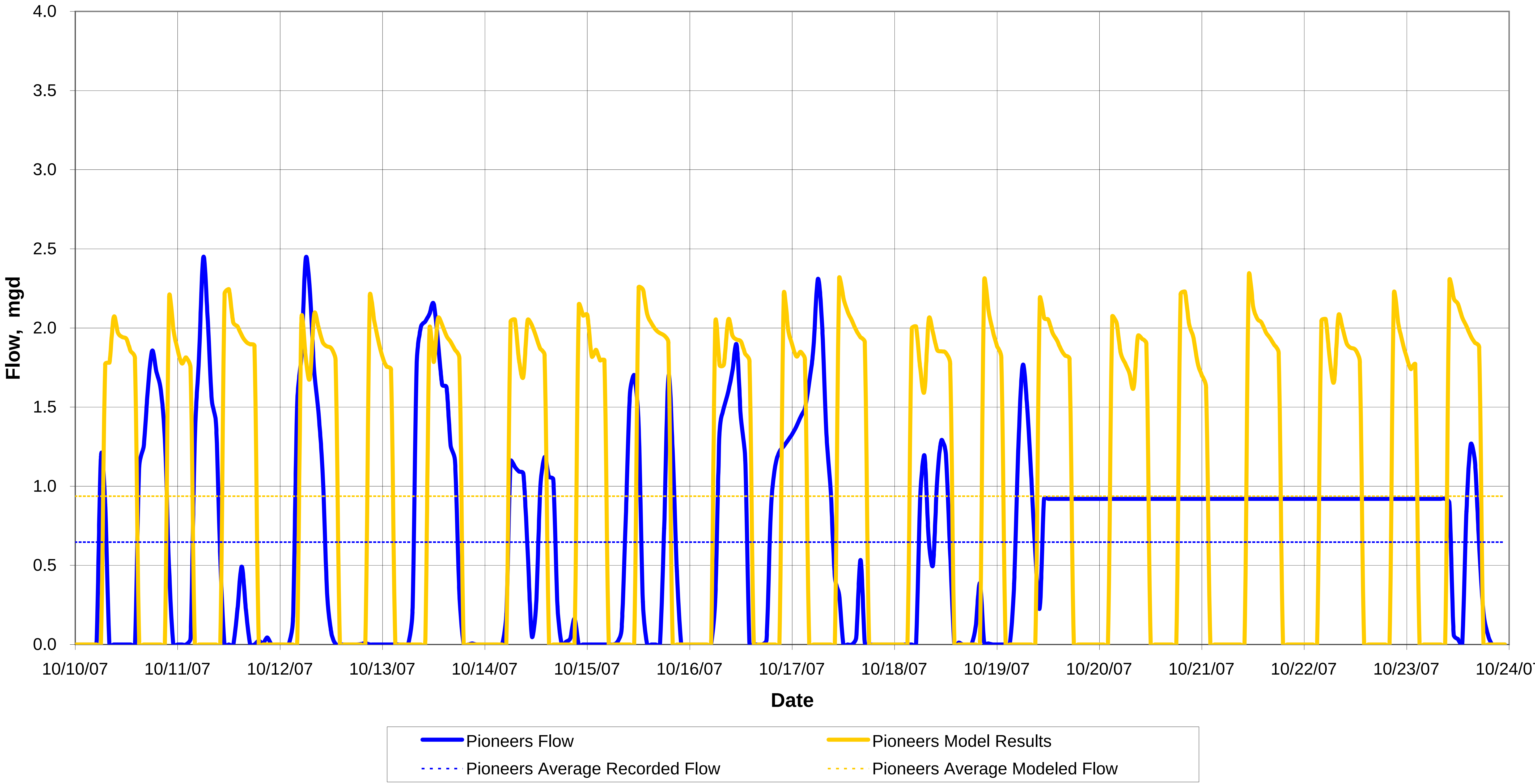

"A" Low Flow
Wednesday October 10th, 2007 - Tuesday October 23rd, 2007
Flow, mgd
20.00
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
10/10/07
10/11/07
10/12/07
10/13/07
10/14/07
10/15/07
10/16/07
10/17/07
10/18/07
10/19/07
10/20/07
10/21/07
10/22/07
10/23/07
10/24/07
Date
"A" Low Flow
"A" Low Model Results
"A" Low Average Recorded Flow
"A" Low Average Modeled Flow


"A" High Flow
Wednesday October 10th, 2007 - Tuesday October 23rd, 2007
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
Flow, mgd
10/10/07
10/11/07
10/12/07
10/13/07
10/14/07
10/15/07
10/16/07
10/17/07
10/18/07
10/19/07
10/20/07
10/21/07
10/22/07
10/23/07
10/24/07
Date
"A" High Flow
"A" High Model Results
"A" High Average Recorded Flow
"A" High Average Modeled Flow

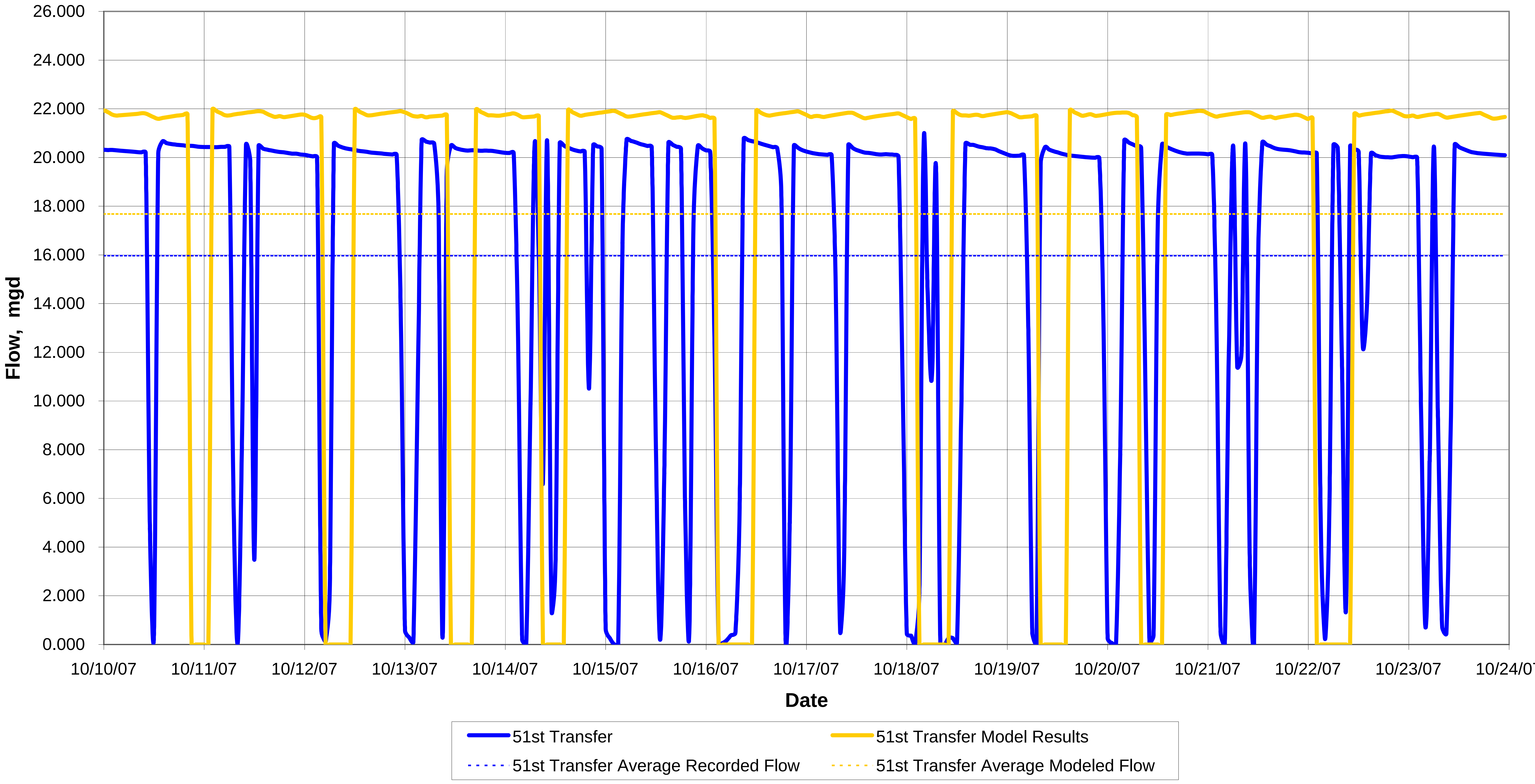


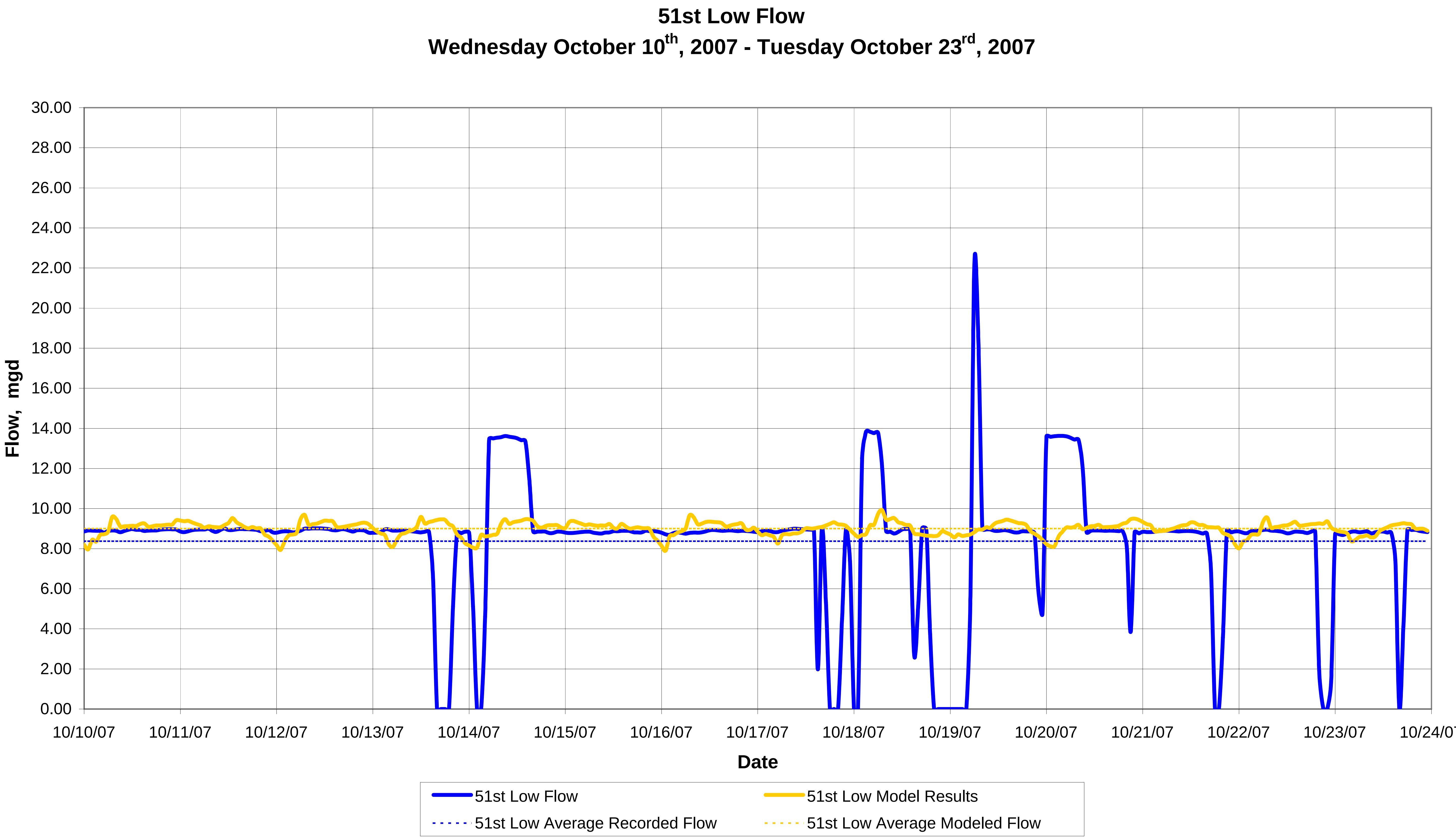
Belmont Flow
Wednesday October 10th, 2007 - Tuesday October 23rd, 2007
Flow, mgd
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
10/10/07
10/11/07
10/12/07
10/13/07
10/14/07
10/15/07
10/16/07
10/17/07
10/18/07
10/19/07
10/20/07
10/21/07
10/22/07
10/23/07
10/24/07
Date
Belmont Flow
Belmont Model Results
Belmont Average Recorded Flow
Belmont Average Modeled Flow

Pioneers Flow
Wednesday October 10th, 2007 - Tuesday October 23rd, 2007

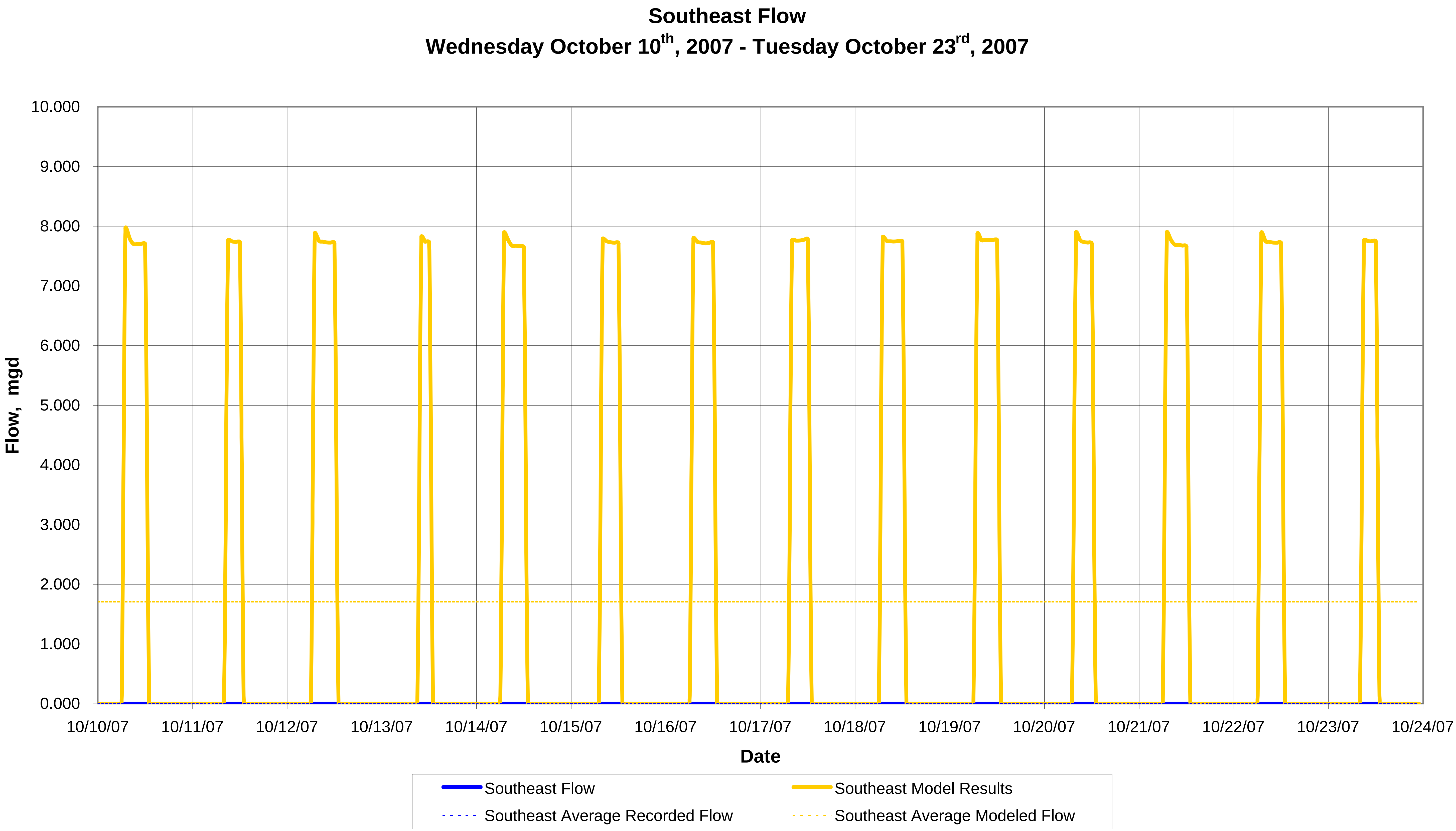


51st Transfer Flow
Wednesday October 10th, 2007 - Tuesday October 23rd, 2007





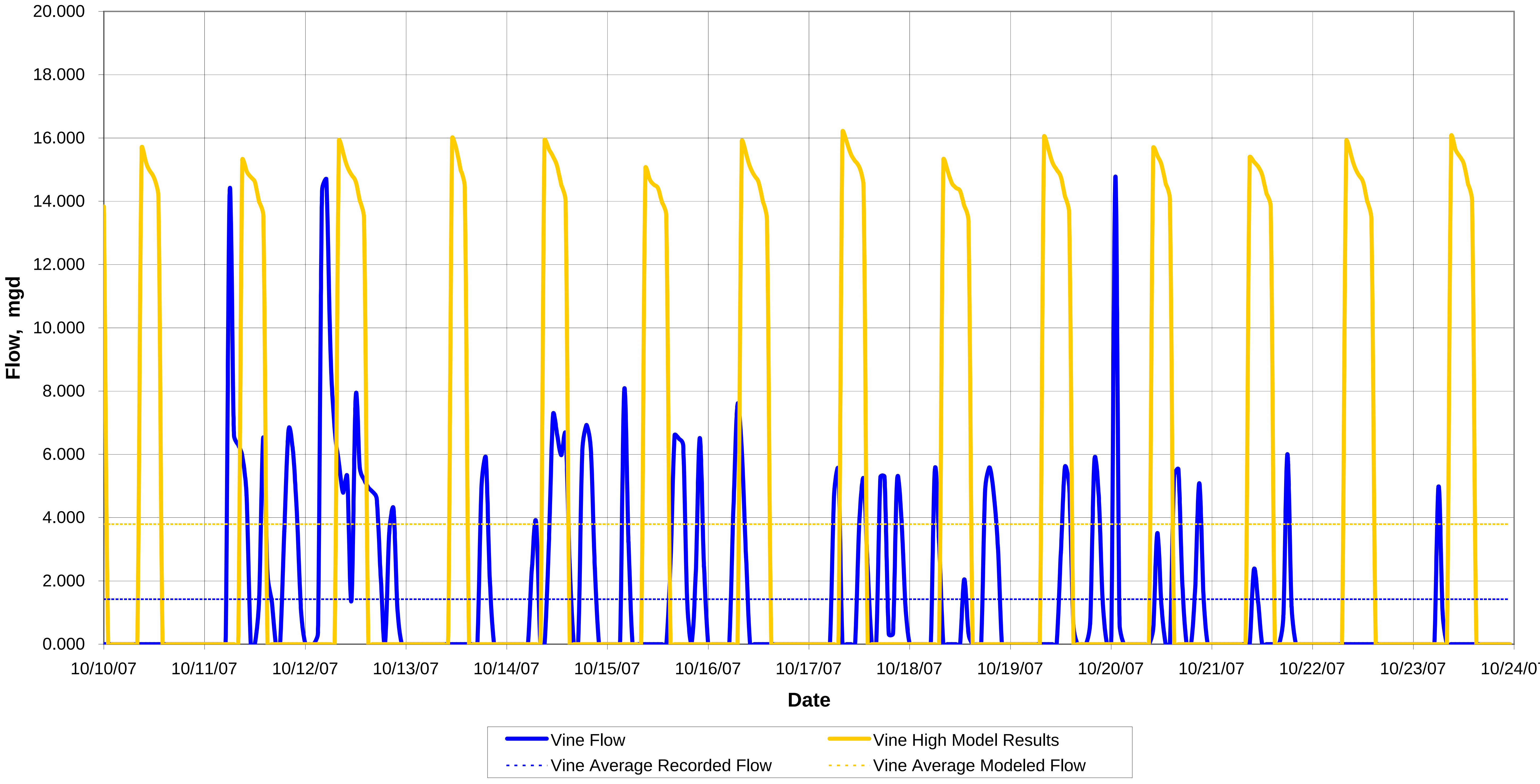
51st Low Flow
Wednesday October 10th, 2007 - Tuesday October 23rd, 2007
Flow, mgd
30.00
28.00
26.00
24.00
22.00
20.00
18.00
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
10/10/07
10/11/07
10/12/07
10/13/07
10/14/07
10/15/07
10/16/07
10/17/07
10/18/07
10/19/07
10/20/07
10/21/07
10/22/07
10/23/07
10/24/07
Date
51st Low Flow
51st Low Model Results
51st Low Average Recorded Flow
51st Low Average Modeled Flow

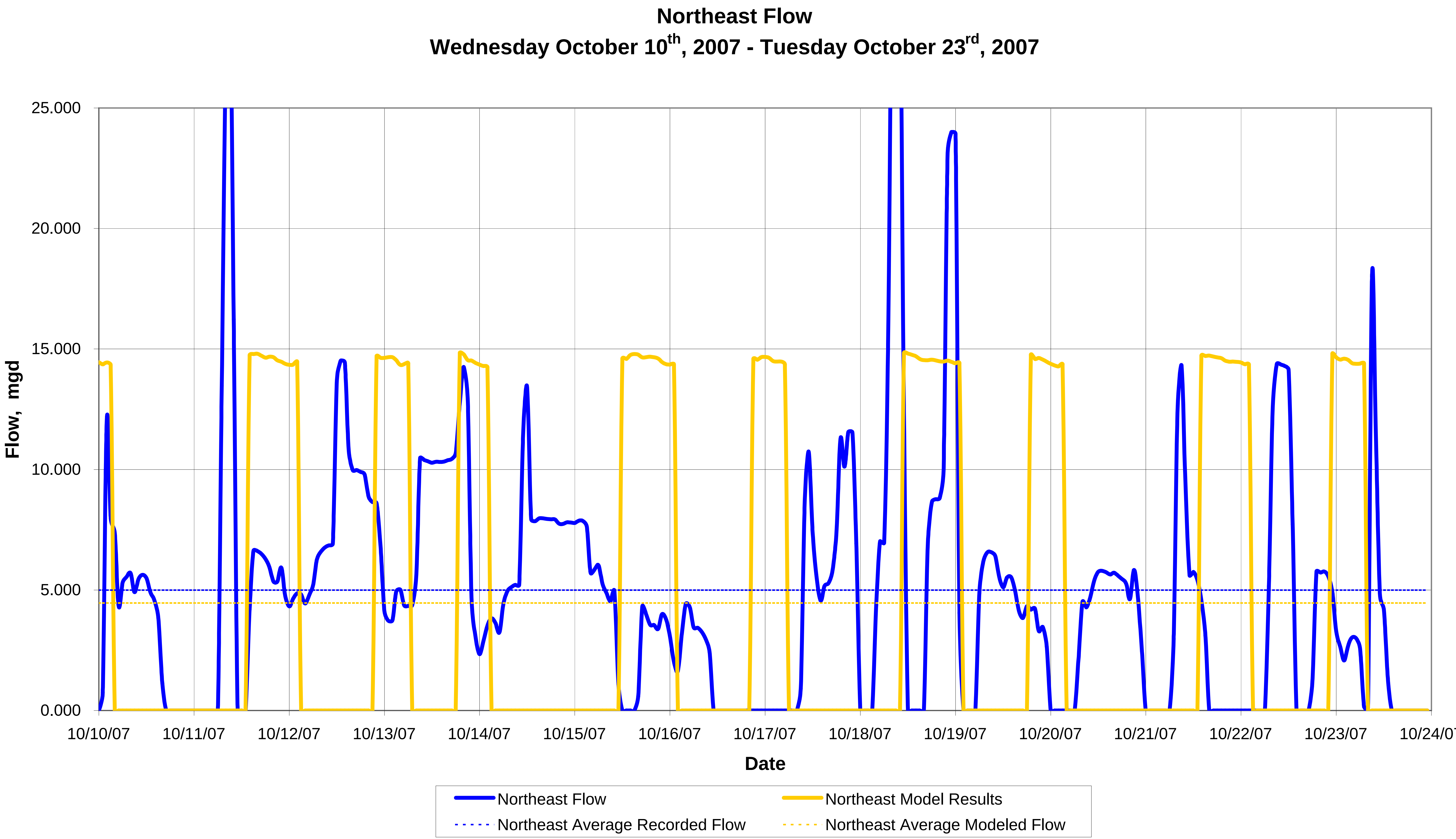


Southeast Flow
Wednesday October 10th, 2007 - Tuesday October 23rd, 2007
10.000
9.000
8.000
7.000
6.000
5.000
4.000
3.000
2.000
1.000
0.000
Flow, mgd
10/10/07
10/11/07
10/12/07
10/13/07
10/14/07
10/15/07
10/16/07
10/17/07
10/18/07
10/19/07
10/20/07
10/21/07
10/22/07
10/23/07
10/24/07
Date
Southeast Flow
Southeast Model Results
Southeast Average Recorded Flow
Southeast Average Modeled Flow

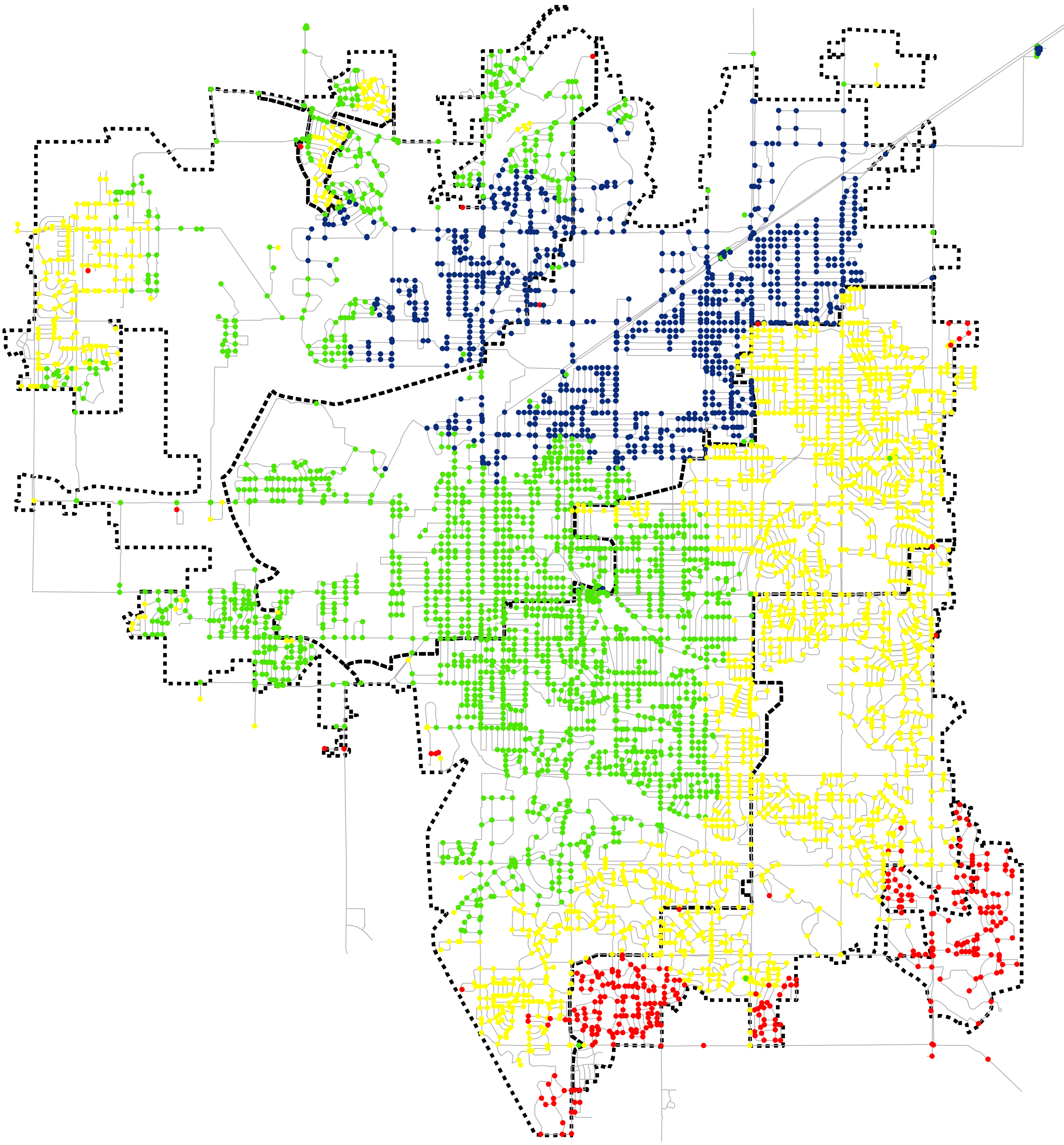
Vine High Flow

Wednesday October 10th, 2007 - Tuesday October 23rd, 2007





Northeast Flow
Wednesday October 10th, 2007 - Tuesday October 23rd, 2007
Flow, mgd
25.000
20.000
15.000
10.000
5.000
0.000
10/10/07
10/11/07
10/12/07
10/13/07
10/14/07
10/15/07
10/16/07
10/17/07
10/18/07
10/19/07
10/20/07
10/21/07
10/22/07
10/23/07
10/24/07
Date
Northeast Flow
Northeast Model Results
Northeast Average Recorded Flow
Northeast Average Modeled Flow



Legend

Pipes

Service Level Boundary

Water Age, hrs

- 0 - 168
- 169 - 336
- 337 - 504
- 505 - 672

N
W E
S

0 3,000 6,000 12,000
Feet

City of Lincoln, Nebraska
Facilities Master Plan Update
2007

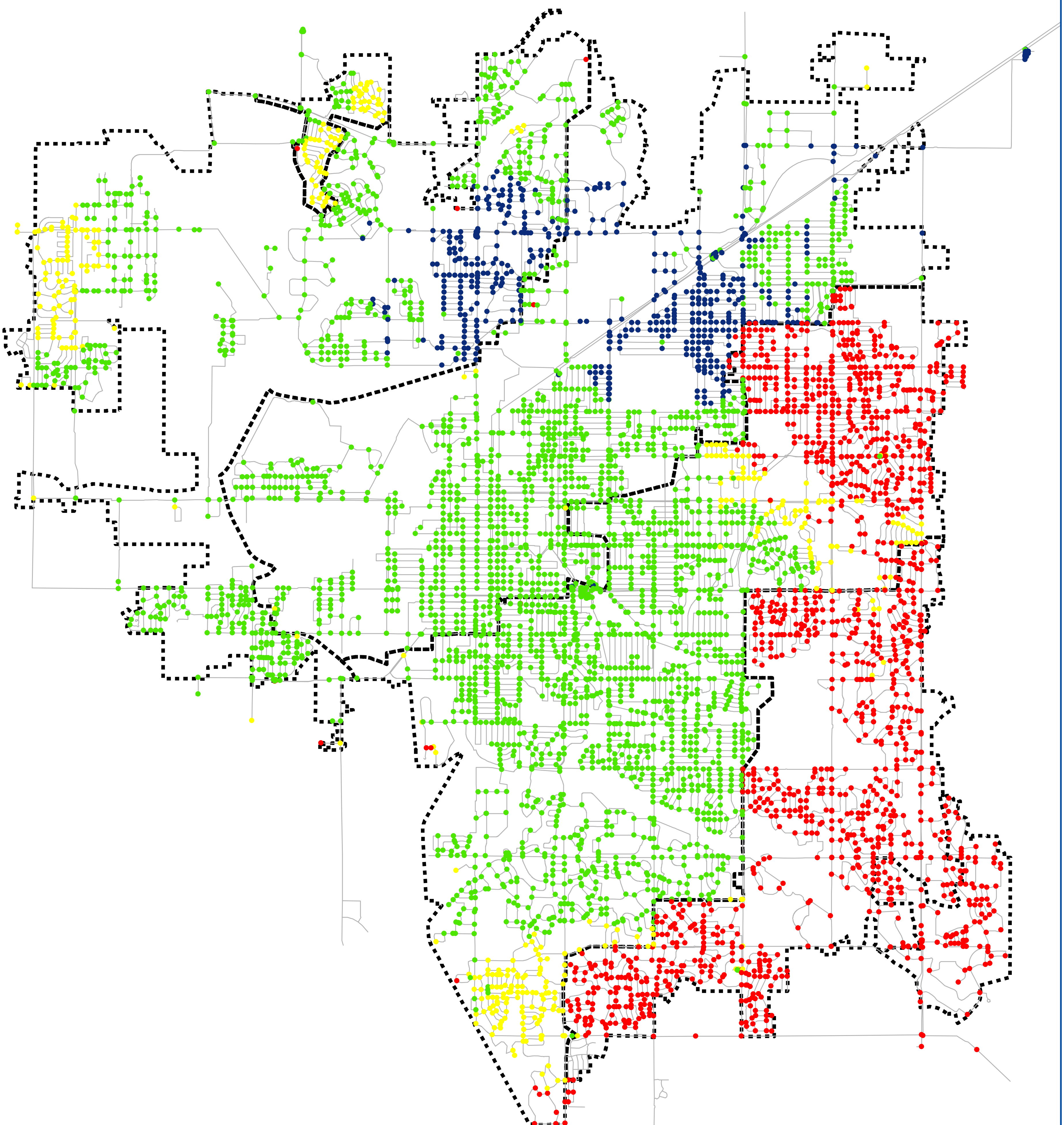
Water Age
Minimum Month Age

FIGURE 1

Lincoln Water System

BLACK & VEATCH
Building a world of difference

Data source: City of Lincoln, B&V
Path: C:\Projects\Lincoln\Phase 500\GIS\AgeMaps.mxd Date Printed: February 14, 2007 Map By: JDM



Legend

Pipes

Service Level Boundary

Water Age, hrs

- 0 - 168
- 169 - 336
- 337 - 504
- 505 - 672

N
W E
S

0 3,000 6,000 12,000 Feet

City of Lincoln, Nebraska
Facilities Master Plan Update
2007

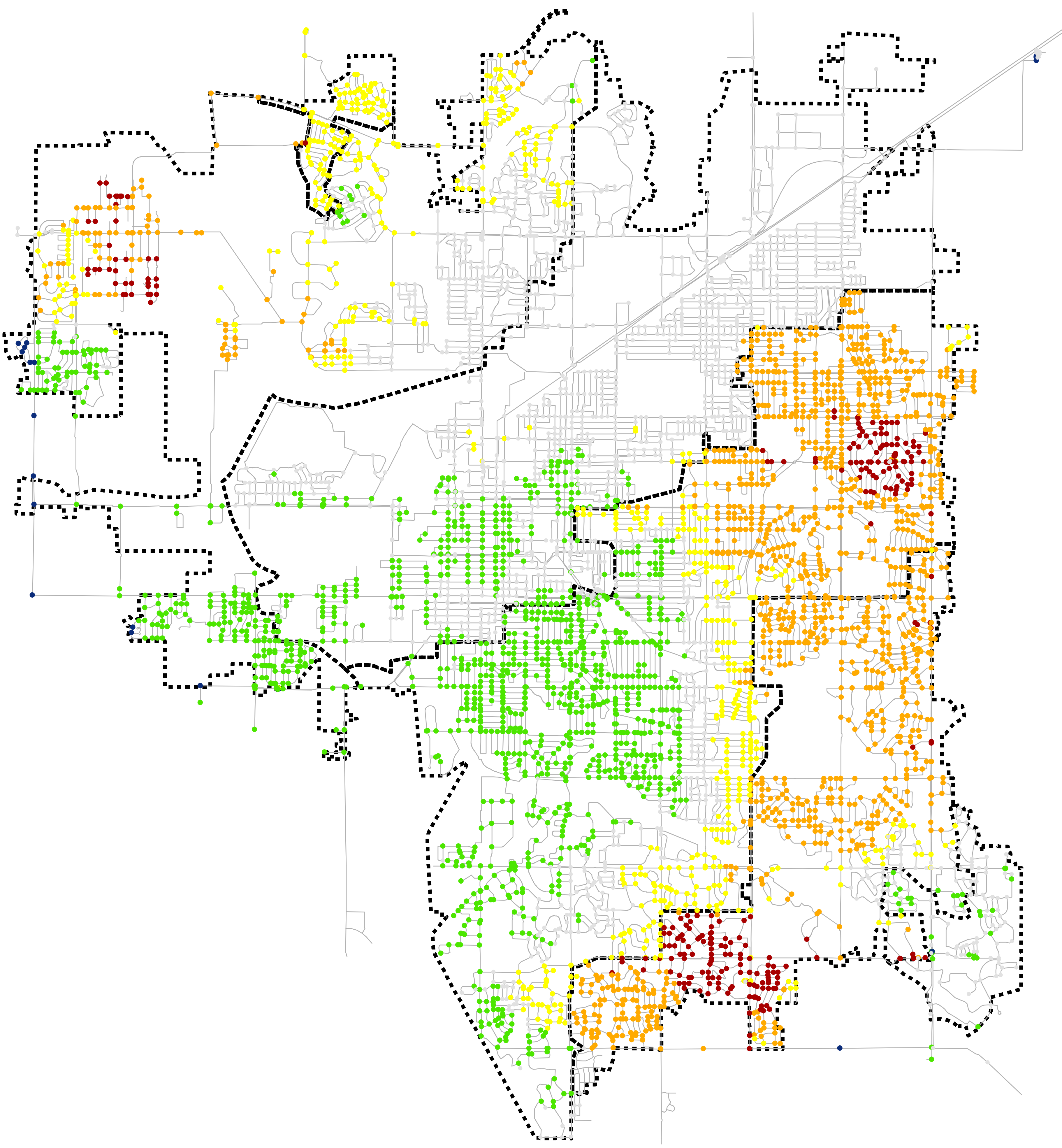
Water Age
Average Month Age

FIGURE 2

Lincoln Water System

BLACK & VEATCH
Building a world of difference.

Data source: City of Lincoln, B&V
Path: C:\Projects\Lincoln\Phase 500\GIS\AgeMaps.mxd Date Printed: February 14, 2007 Map By: JDM



Legend

Pipes

Service Level Boundary

Water Age Difference, hrs

- <-100
- -99 - -50
- -49 - -10 **Older**
- -9 - 10 **(No Change)**
- 11 - 50 **Fresher**
- > 50

N
W E
S

0 3,000 6,000 12,000
Feet

City of Lincoln, Nebraska
Facilities Master Plan Update
2007

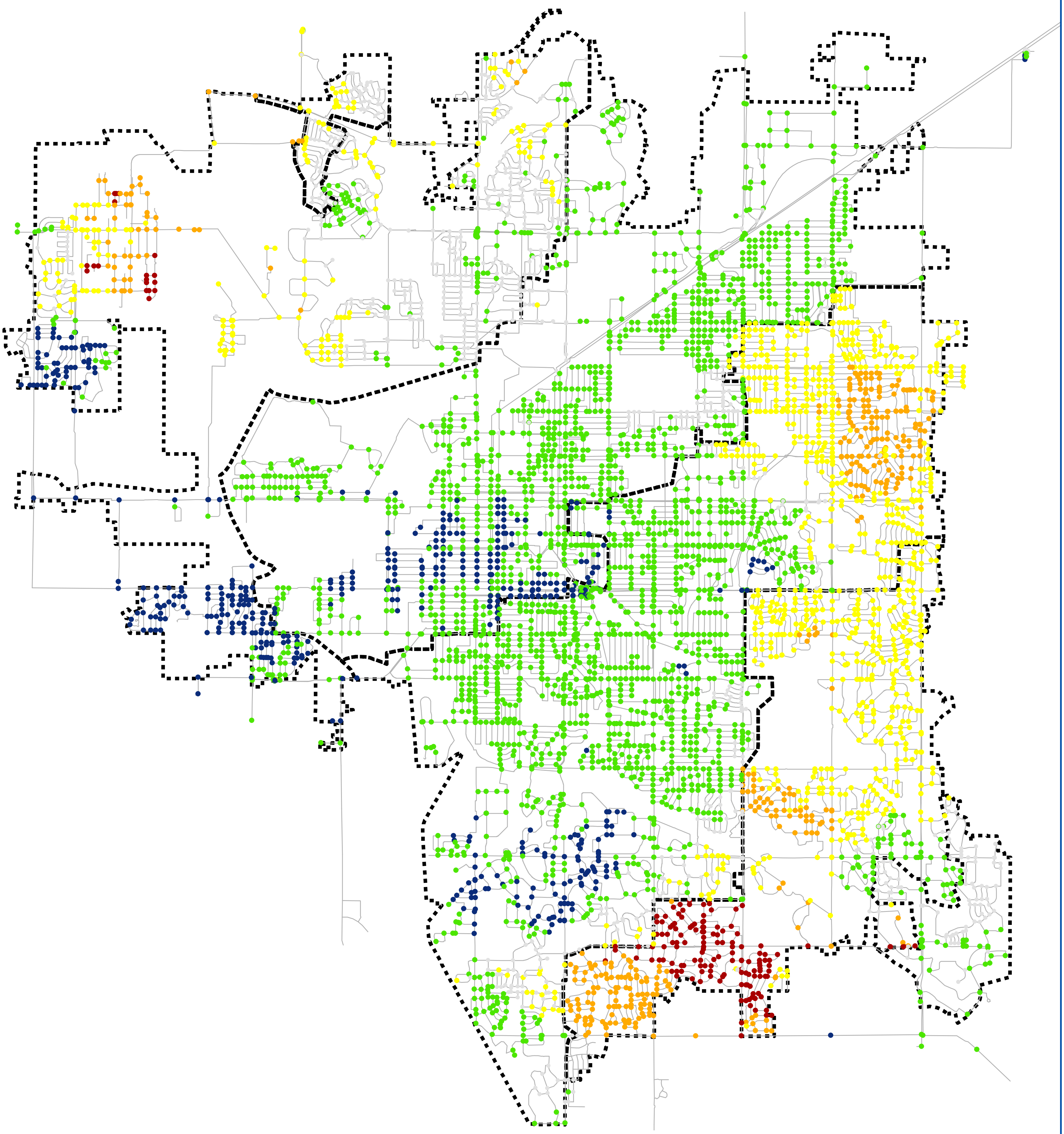
Water Age Difference
(MIN_MONTH_BASE - MIN_MONTH_ALT2)

FIGURE 3

Lincoln Water System

BLACK & VEATCH
Building a world of difference®

Data source: City of Lincoln, B&V
Path: C:\Projects\Lincoln\Phase 500\GIS\AgeMaps.mxd Date Printed: February 14, 2007 Map By: JDM



Legend

Pipes

Service Level Boundary

Water Age Difference, hrs

- <-100
- -99 - -50
- -49 - -10 — **Older**
- -9 - 10 — **(No Change)**
- 11 - 50 — **Fresher**
- > 50

N W E S

0 3,000 6,000 12,000 Feet

City of Lincoln, Nebraska
Facilities Master Plan Update
2007

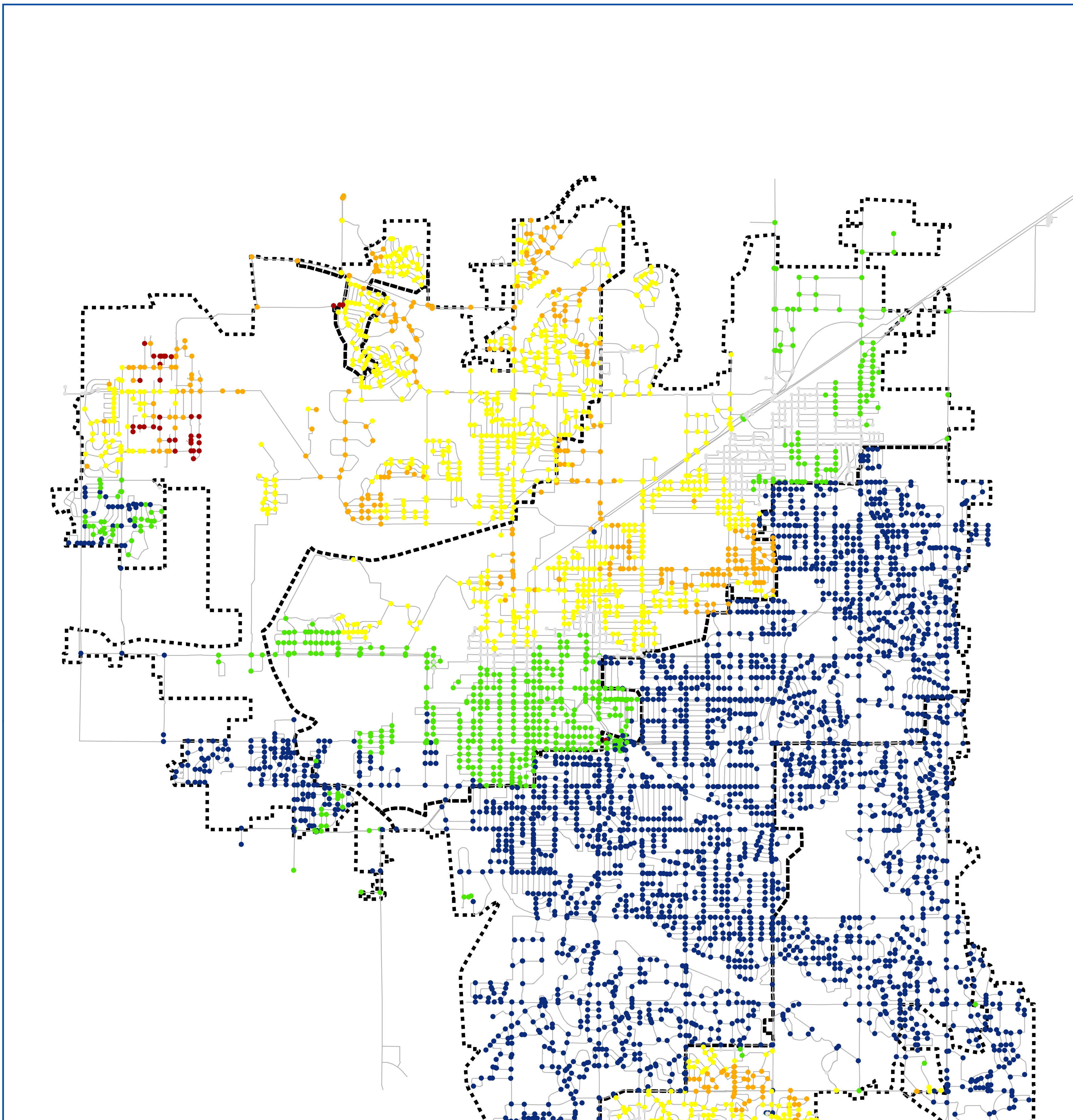
Water Age Difference
(MIN_MONTH_BASE - MIN_MONTH_ALT4)

FIGURE 4

Lincoln Water System

BLACK & VEATCH Building a world of difference

Data source: City of Lincoln, B&V
Path: C:\Projects\Lincoln\Phase 500\GIS\AgeMaps.mxd Date Printed: February 14, 2007 Map By: JDM



Legend

Pipes

Service Level Boundary

Water Age Difference, hrs

- <-100
- -99 - -50
- -49 - -10
- -9 - 10
- 11 - 50
- > 50

Older

(No Change)

Fresher

N

W E

S

0 3,000 6,000 12,000
Feet

City of Lincoln, Nebraska
Facilities Master Plan Update
2007

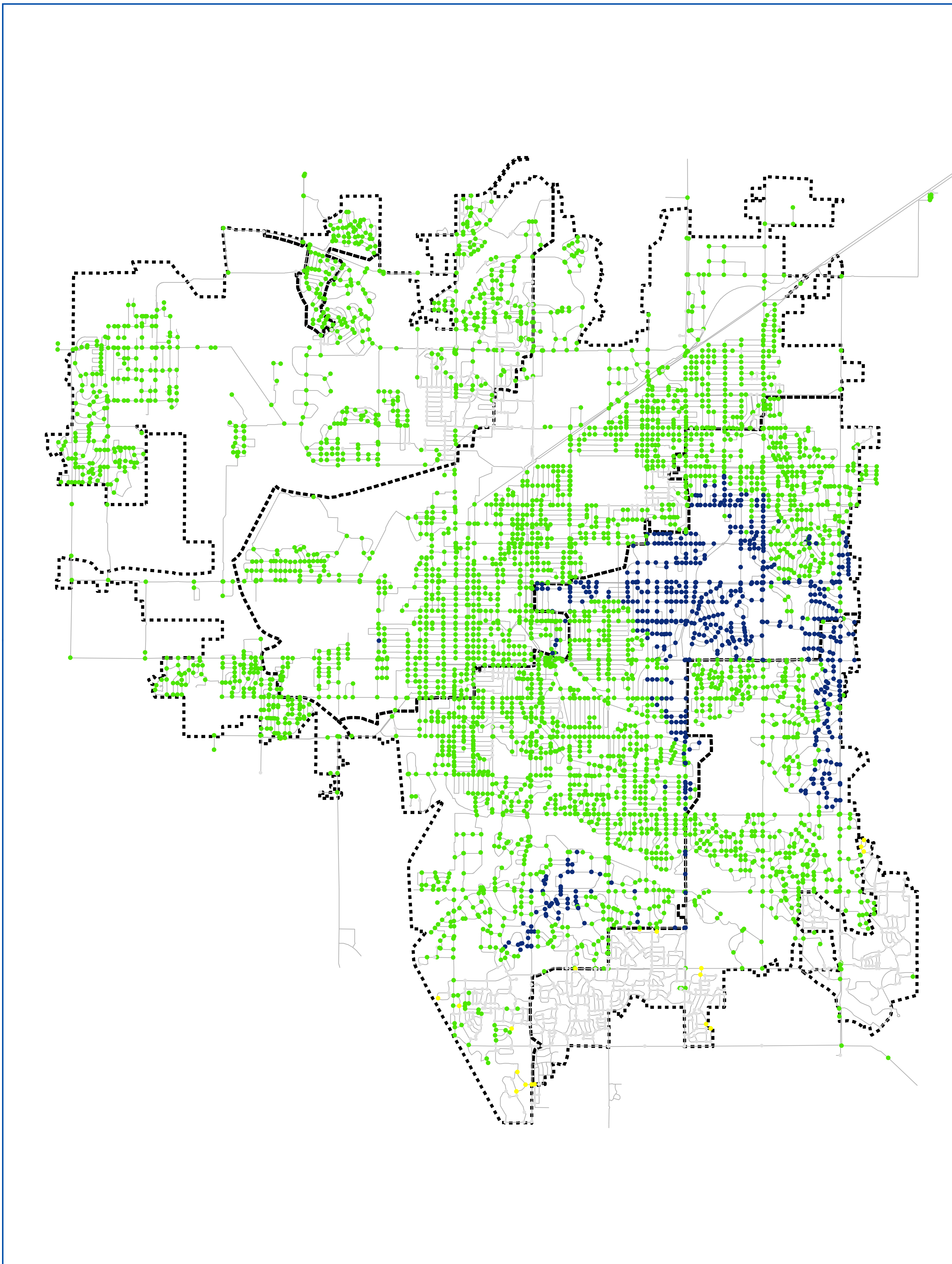
Water Age Difference
(MIN_MONTH_BASE - MIN_MONTH_ALT5)

FIGURE 5

Lincoln Water System

BLACK & VEATCH
Building a world of difference

Data source: City of Lincoln, B&V
Path: C:\Projects\Lincoln\Phase 500\GIS\AgeMaps.mxd Date Printed: February 14, 2007 Map By: JDM



Legend

Pipes

Service Level Boundary

Water Age Difference, hrs

- -176 - -100
- -99 - -50
- -49 - -10 **Older**
- -9 - 10 **(No Change)**
- 11 - 50 **Fresher**
- 51 - 373

N
W E
S

0 3,000 6,000 12,000
Feet

City of Lincoln, Nebraska
Facilities Master Plan Update
2007

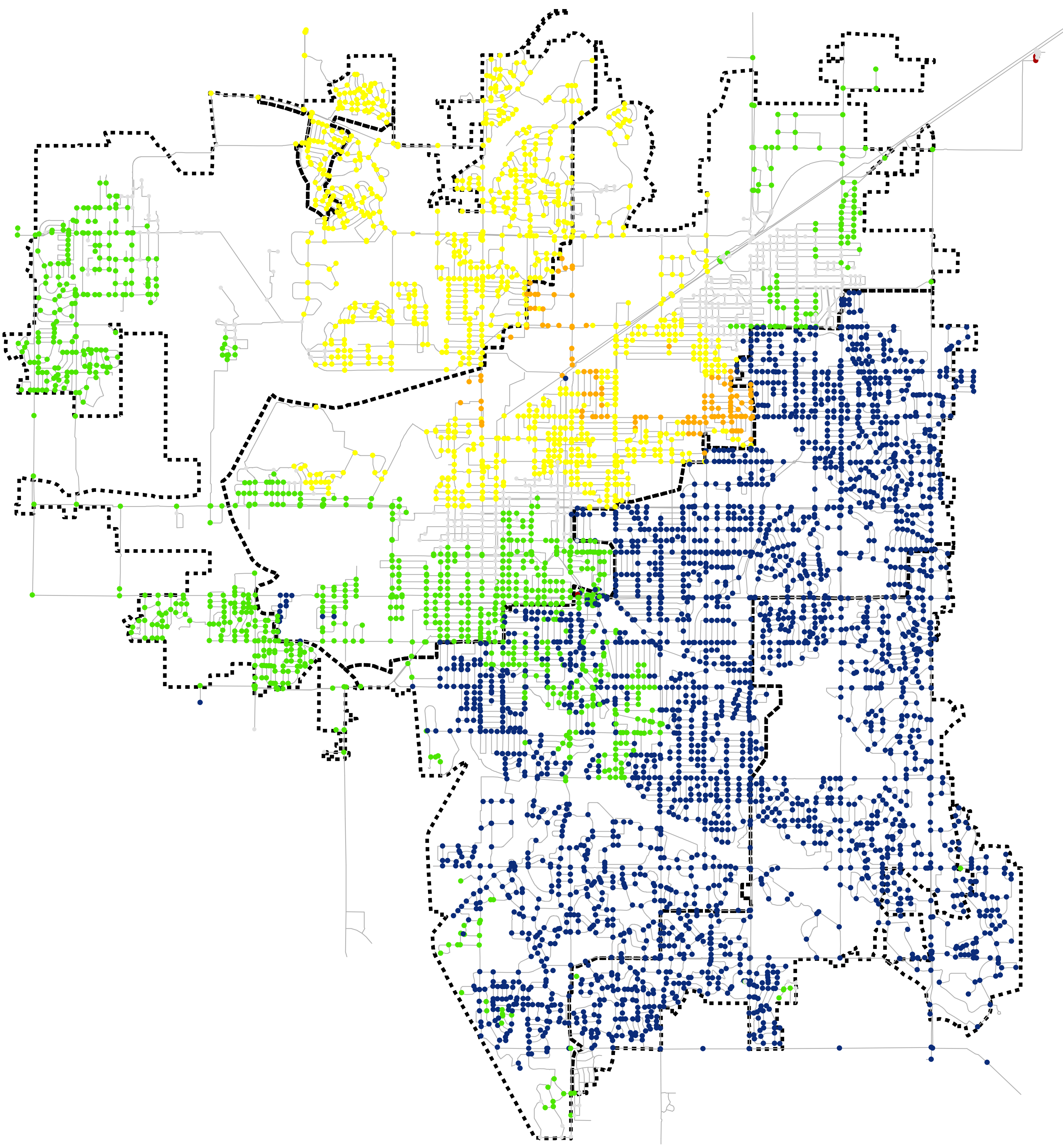
Water Age Difference
(MIN_MONTH_ALT2 - MIN_MONTH_ALT4)

FIGURE 6

Lincoln Water System

BLACK & VEATCH
Building a world of difference

Data source: City of Lincoln, B&V
Path: C:\Projects\Lincoln\Phase 500\GIS\AgeMaps.mxd Date Printed: February 14, 2007 Map By: JDM



Legend

Pipes

Service Level Boundary

Water Age Difference, hrs

- <-100
- -99 - -50
- -49 - -10
- -9 - 10
- 11 - 50
- > 50

Older

(No Change)

Fresher

N
W E
S

0 3,000 6,000 12,000
Feet

City of Lincoln, Nebraska
Facilities Master Plan Update
2007

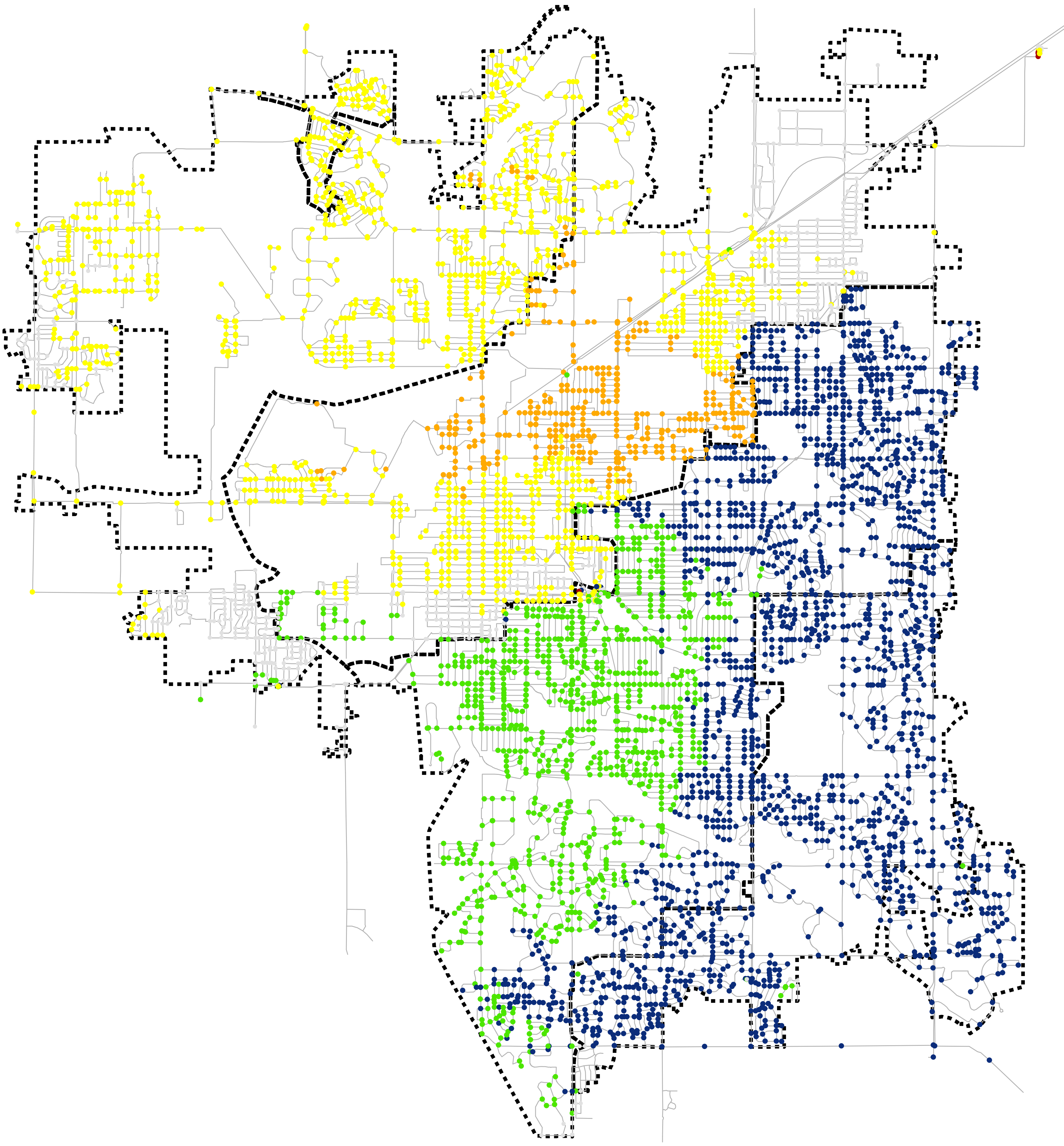
Water Age Difference
(MIN_MONTH_ALT2 - MIN_MONTH_ALT5)

FIGURE 7

Lincoln Water System

BLACK & VEATCH
Building a world of difference

Data source: City of Lincoln, B&V
Path: C:\Projects\Lincoln\Phase 500\GIS\AgeMaps.mxd Date Printed: February 14, 2007 Map By: JDM



Legend

Pipes

Service Level Boundary

Water Age Difference, hrs

- <-100
- -99 - -50
- -49 - -10 **Older**
- -9 - 10 **(No Change)**
- 11 - 50 **Fresher**
- > 50

N
W E
S

0 3,000 6,000 12,000
Feet

City of Lincoln, Nebraska
Facilities Master Plan Update
2007

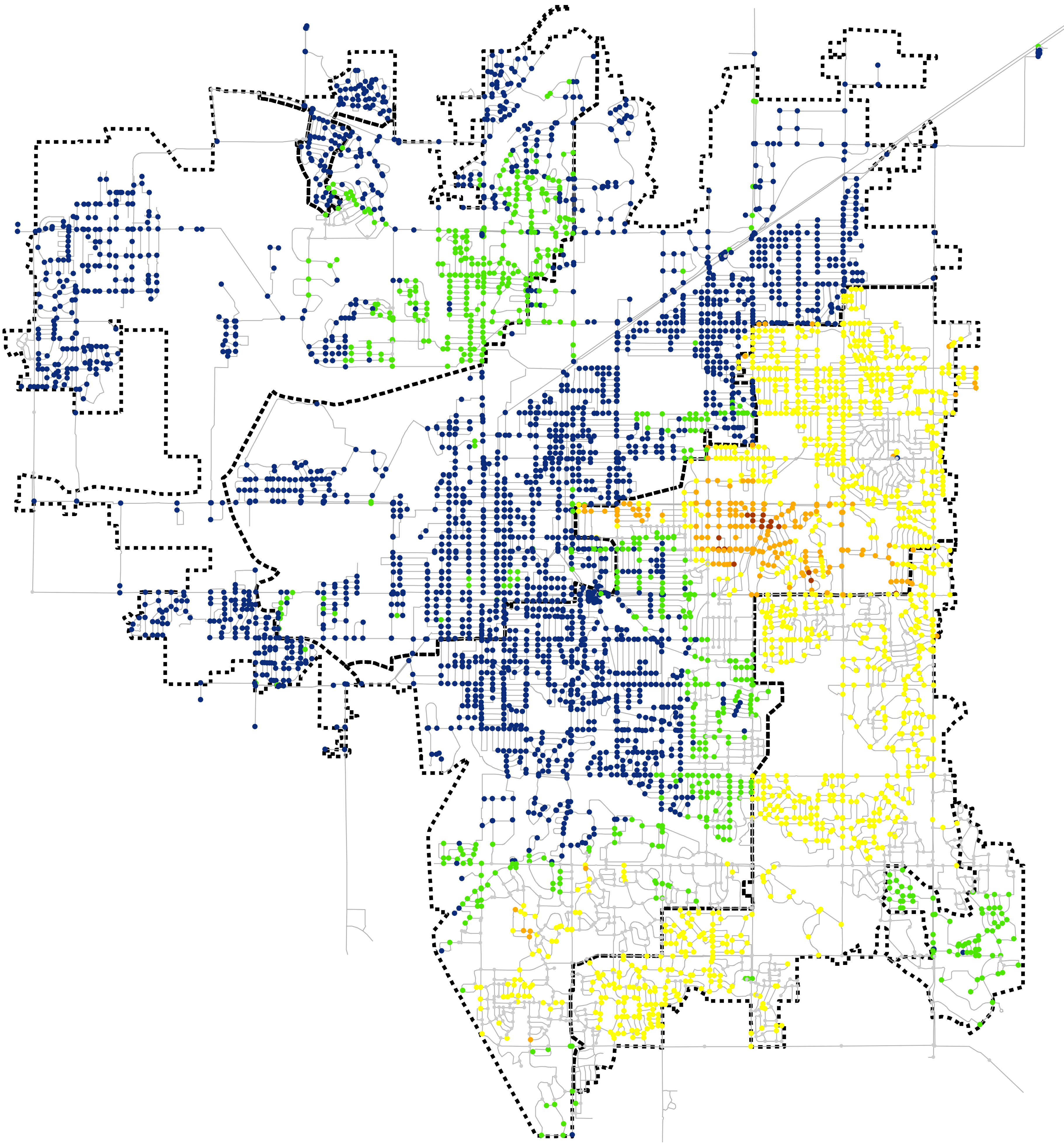
Water Age Difference
(MIN_MONTH_ALT4 - MIN_MONTH_ALT5)

FIGURE 8

Lincoln Water System

BLACK & VEATCH
Building a world of difference

Data source: City of Lincoln, B&V
Path: C:\Projects\Lincoln\Phase 500\GIS\AgeMaps.mxd Date Printed: February 14, 2007 Map By: JDM



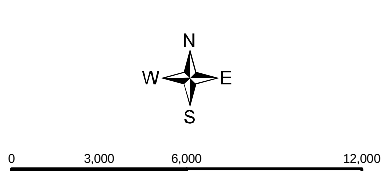
Legend

Pipes

Service Level Boundary

Water Age Difference, hrs

●	-97 - -30	
●	-29 - -20	
●	-19 - -10	**Older**
●	-9 - 10	**(No Change)**
●	11 - 20	**Fresher**
●	21 - 154	



City of Lincoln, Nebraska
Facilities Master Plan Update
2007

Water Age Difference
(MIN_MONTH_BASE - MIN_MONTH_OCT_DMND)

FIGURE 9

Lincoln Water System

BLACK & VEATCH
Building a world of difference

Data source: City of Lincoln, B&V
Path: C:\Projects\Lincoln\Phase 500\GIS\AgeMaps.mxd Date Printed: February 14, 2007 Map By: JDM