CITY OF MILWAUKIE

2010 Water System Master Plan

WEST YOST ASSOCIATES

MILWAUKIE, OREGON · FOUNDED 1847

2010 Water System Master Plan

Prepared for

City of Milwaukie

WEST YOST ASSOCIATES

382-03-10-01

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ES.1 INTRODUCTION

This 2010 Water System Master Plan for the City of Milwaukie (City) identifies strategies for maintaining adequate water supplies and service levels for the community; guides capital expenditures for the system; furnishes important guidance on operational issues; and charts a course for future updates to water rates. To accomplish these goals, the following work tasks were performed in the WSMP:

- Evaluate and summarize existing water system and key system facilities;
- Develop water demand projections through buildout;
- Evaluate existing and future water supplies to develop a strategy for the City to meet existing and future water demands;
- Develop performance and operational criteria under which the water system will be analyzed and future facilities will be formulated;
- Develop and calibrate a water distribution system hydraulic model;
- Evaluate existing and buildout water system conditions to identify the City's water distribution system facility needs; and,
- Develop a capital improvement program for recommended existing and future water system facilities.

A summary of the key work tasks is provided below. Complete descriptions of all the analyses and assessments are provided in the following chapters and appendices of this Water System Master Plan.

ES.2 OVERVIEW OF THE CITY'S SERVICE AREA

A detailed description of the City's existing service area is provided in Chapter 2. The following subsections present a brief overview of the City's service area.

ES.2.1 City of Milwaukie Service Area

The City currently provides potable water service to most areas within the City limits, though some residents are served by other providers. The City is located in the Portland Metropolitan Area approximately 7 miles south of downtown Portland and is bounded on the west by the Willamette River, the north roughly by Johnson Creek Boulevard, the east roughly by Linwood Avenue and 71st Avenue and the south by Kellogg Creek and Lake Road. The City limits and service area are shown in Figure ES-1 and include approximately 3,169 acres, or about 4.95 square miles.

The City has a mix of users including various density residential areas, commercial, industrial and public. The City also has a town center designation in the old downtown area near the Willamette River. The City is approximately 97 percent developed and there are few vacant areas within the City limit.

In 1990, approximately 7,400 acres of land adjacent to the City was designated as an Urban Growth Management Area (UGMA). This means that these lands will be the first areas of growth for the region. Being that they are adjacent to the City, some, or all, of the areas could be annexed into the City. Based on the vacant land inventory, this area is also highly developed and only five percent (395 acres) of the area is currently vacant.

Within the UGMA lies a subset of land known as Dual Interest Areas A and B. These areas are currently almost entirely surrounded by the City, but are being served by Clackamas River Water District (CRW) (see Figure ES-2). These are areas that may come into the City's service area in the near future, and therefore need careful planning and consideration.

ES.2.2 Hydrology and Water Sources

The City gets its water from the underground basin of the Troutdale Aquifer. The Troutdale Aquifer is approximately 300 square miles and is part of the larger Portland Basin that includes portions of the States of Oregon and Washington. The aquifer is synclinal and the center of the basin is well-confined by low-permeability layers making it a good municipal source. The City has interconnections with the City of Portland and the CRW. These are used only in the event of an emergency.

ES.2.3 Population Served

According to the Portland State University Population Research Center, the City has a 2009 estimated population of 20,920. The average annual population growth during the last decade has been 0.20 percent. Since growth in the City is through infill, growth has been much slower than neighboring communities with available land for larger scale development.

ES.3 OVERVIEW OF THE EXISTING WATER SYSTEM

A detailed description of the City's existing water system is provided in Chapter 3. The following subsections present a brief overview of the City's existing water system.

ES.3.1 Water Supply

The City relies entirely on groundwater for its base water supply and has two emergency interties, one with the City of Portland and one with the CRW, both surface water systems. Groundwater from two City-owned wells is pumped directly into the distribution system while water from the other five wells is treated before it is pumped into the distribution system. The City's wells pump from the Troutdale Formation that is an extensive aquifer underlying the Portland Metropolitan Area and a large portion of Clark County, Washington. This aquifer is a deep system of gravels and sandstone with large unconsolidated areas. All of the City's wells have active water rights that are certified through the Oregon Water Resources Department.

As summarized in the most recent water quality report, water quality for the City surpasses all state and federal standards for drinking water. While Volatile Organic Compounds (VOCs) are present in five of the City's wells, sampling shows that after treatment the VOCs are not present in delivered water.

ES.3.2 Water System Facilities

The City's existing system facilities consist of wells, treatment facilities, storage reservoirs, pump stations and pressure reducing valves (PRVs). Locations of these facilities are shown on Figure ES-2.

The City has eight wells of which seven are operational. Well No. 1 is off line with capacity used by Wells 2, 3, and 5. Wells 2 through 8 have a combined permitted production capacity of 5,094 gallons per minute (gpm) or 7.3 million gallons per day (mgd).

The City operates two treatment facilities that have the same configuration and general operating procedures. Due to VOCs found in Well Nos. 2, 3, 4, 5, and 7, air stripping towers were installed for these wells in 1990.

The City currently operates one elevated steel reservoir (Elevated Reservoir), one ground level steel reservoir (Stanley Reservoir) and one ground level concrete reservoir (Concrete Reservoir). The City has a total above-ground storage capacity of 6.0 million gallons (MG).

The City maintains two transfer pump stations and two booster pump stations.

There are approximately 112 miles of pipeline in the City that range in size from 1 to 18 inches in diameter.

ES.3.3 Pressure Zones

The City water distribution system has four pressure zones as shown on Figure ES-2. Zones 1 and 2 are fed by gravity from storage reservoirs and range in elevation from 28 to 125 feet and 50 to 195 feet respectively. Zones 3 and 4 are both fed from constant pumping stations. Zone 3 ranges from 160 to 205 feet in elevation and Zone 4 ranges from 75 to 150 feet in elevation.

The City operates several pressure reducing stations to manage water pressure between zones.

ES.4 EXISTING AND FUTURE WATER DEMANDS

A detailed description of the City's existing and projected future water demands is provided in Chapter 4. The following subsections present a brief overview of existing and future water demands for the City.

ES.4.1 Existing Water Demands

The City measures all of the water produced by its wells, received from adjacent water purveyors, and meters all of its customers. Consequently, the City tracks water use in two ways: production records and meter (consumption) records.

Existing water demands for the City were determined based on historical water production and historical consumption data. The historical average per capita water demand has remained relatively stable, averaging about 116 gallons per capita per day (gpcd) over the past 10 years. On average, the City uses about 2.4 mgd.

Peaking factors are used to calculate water demands expected under high demand conditions (*i.e.*, maximum day and peak hour demand). The resulting demand conditions for maximum day and peak hour periods are then used to evaluate and size transmission/distribution pipelines and storage facilities, and to define water supply needs and capacity requirements. Peaking factors for maximum day and peak hour demand were developed based on historical production records and are shown in Table ES-1.

Table ES-1. Adopted Peaking Factors

Type of Factor	Adopted Factor
Average Day (ADD) to Maximum Day Demand (MDD)	1.9
Average Day to Peak Hour Demand (PHD)	2.7

ES.4.2 Projected Water Demands

Water demands were projected through buildout of the City using a unit demand methodology based on land uses in the Comprehensive Plan. A land use based methodology was used instead of a per capita demand methodology, because per capita water demand projections uniformly distribute water use over the entire water service area and therefore, do not account for specific land uses and associated water demands in specific locations.

Table ES-2 summarizes the current and buildout demands for the City’s current service area, the dual interest areas and the UGMA.

As shown in Table ES-2, the buildout demand for the existing service area will only increase by about four percent since most of the area is developed. Serving both Dual Interest Area A and B will add an average demand of 300,000 gpd or 12 percent.

Water demand for the UGMA would more than double the existing water demand in the City. As shown in Table ES-2, the existing average demand in the City is 2.4 mgd, while the demand for the UGMA has been estimated at 4.2 mgd. This average demand is based on land use and has not been confirmed through an analysis of the billing records for CRW.

Table ES-2. Water Demand Projections, mgd										
Demand	Current Service Area		Dual Interest Area A		Dual Interest Area B		UGMA		Total	
	2009	Buildout	2009	Buildout	2009	Buildout	2009	Buildout	2009	Buildout
Average Day	2.4	2.5	0.2	0.2	0.1	0.1	4.2	4.5	6.8	7.3
Maximum Day	4.6	4.8	0.3	0.3	0.2	0.2	7.9	8.6	13.0	13.9
Peak Hour	6.5	6.8	0.4	0.5	0.3	0.3	11.2	12.2	18.4	19.8

ES.5 HYDRAULIC MODEL DEVELOPMENT

To develop the City's hydraulic network model, West Yost completed the following steps:

- Used the City's existing water distribution system maps (exported from the City's GIS) to create the hydraulic model;
- Verified that the hydraulic model system configuration (pipeline sizes, alignments, connections, and other facility sizes and locations) is representative of the City's current water system;
- Allocated existing water demands by using City's spatially located account information to distribute demands within the hydraulic model; and
- Calibrated the City's water system hydraulic model to simulate pressures and flows observed in the field.

A detailed description of the development, calibration and verification of the City's water distribution system hydraulic model is provided in Chapter 6.

ES.6 EXISTING WATER SYSTEM

A detailed description of the evaluation the existing water system is provided in Chapter 7. The following subsections presents a brief overview of the evaluation and recommended improvements for the existing water system.

ES.6.1 Existing Water System Evaluation and Recommended Improvements

ES.6.1.1 Water Storage Capacity

The City currently has 6.0 MG of water storage, which is sufficient for the existing water system.

ES.6.1.2 Pumping Capacity

The City currently has a firm pumping capacity deficiency of 1,723 gpm in Pressure Zone 3. The addition of two 1,750 gpm fire flow pumps to the pump station in Zone 3 will resolve this deficiency.

ES.6.1.3 Water Distribution System

During a peak hour demand condition, results indicate that the existing system in Pressure Zones 2, 3, and 4 can adequately deliver peak hour demands under the City's minimum pressure criteria of 40 psi.

During a peak hour demand condition, results indicate that the existing system in Pressure Zone 1 can adequately deliver peak hour demands to most of the Zone under the City's minimum pressure criteria of 40 psi. Most of the locations with pressures below 40 psi are either within a few psi of the acceptable range, or are located above the elevation that will support a 40 psi pressure given the HGL of Pressure Zone 1. There are two locations to the West of the Zone 1/Zone 2 boundary, adjacent to the intersection of Sparrow Street and 22nd Avenue, with pressures of 34 psi that are located at an elevation that could meet the 40 psi criterion if pipes were sized adequately. It is recommended that the current location of pressure zone breaks be evaluated and adjustments made to eliminate the existing deficiencies.

During a maximum day plus fire flow demand scenario, results indicate that many areas in the existing water system in Pressure Zones 1, 2, and 3 could not maintain a minimum system pressure of 20 psi under the required fire flow.

To improve fire flows throughout the service area, the following improvements are recommended:

<u>Fire Flow Improvements in Areas Zoned "Public"</u>

- Upsize approximately 320 feet of existing 6-inch diameter pipeline to 8-inch diameter pipeline from the hydrant to Willard Street in Zone 1.
- Upsize approximately 600 feet of existing 6-inch diameter pipeline to 8-inch diameter pipeline and upsize approximately 95 feet of existing 4-inch diameter pipeline to 8-inch diameter pipeline in the area to the west of Flavel Drive in Zone 2.

<u>Fire Flow Improvements to 4" Pipelines Constructed Prior to 1960</u>

- Replace approximately 10 lineal feet of 4-inch diameter pipeline in pressure Zone 1 with 8-inch diameter pipeline, see Figure ES-3.
- Replace approximately 10,582 lineal feet of 4-inch diameter pipeline in pressure Zone 2 with 8-inch diameter pipeline, see Figure ES-3.
- Replace approximately 2,975 lineal feet of 4-inch diameter pipeline in pressure Zone 3 with 8-inch diameter pipeline, see Figure ES-3.

<u>Fire Flow Improvements to 6" Pipelines Constructed Prior to 1960</u>

- Replace approximately 15,156 lineal feet of 6-inch diameter pipeline in pressure Zone 1 with 8-inch diameter pipeline, see Figure ES-3.
- Replace approximately 49,373 lineal feet of 6-inch diameter pipeline in pressure Zone 2 with 8-inch diameter pipeline, see Figure ES-3.
- Replace approximately 5,329 lineal feet of 6-inch diameter pipeline in pressure Zone 3 with 8-inch diameter pipeline, see Figure ES-3.
- Replace approximately 361 lineal feet of 6-inch diameter pipeline in pressure Zone 4 with 8-inch diameter pipeline, see Figure ES-3.
- Improve fire flow capacity in the existing water system as part of future pipeline replacement projects.

ES.7 FUTURE WATER SYSTEM EVALUATION

A detailed description of the evaluation of the future water system is provided in Chapter 8. The following subsections present a brief overview of the evaluation and recommended improvements for the City's future water system. This evaluation assumed that all recommendations made in the existing system chapter (Chapter 3) have been implemented.

ES.7.1 Buildout Water System Evaluation and Recommended Improvements

ES.7.1.1 Water Storage Capacity

The City currently has 6.0 MG of water storage, which is sufficient to accommodate buildout demand.

ES.7.1.2 Pumping Capacity

The pumping capacity analysis indicates that the City has a pumping capacity surplus of 1,219 gpm for the buildout system.

ES.7.1.3 Water Distribution System

During a peak hour demand condition, results indicate that the buildout system in Pressure Zones 2, 3, and 4 can adequately deliver peak hour demands under the City's minimum pressure criteria of 40 psi.

During a peak hour demand condition, results indicate that the buildout system in Pressure Zone 1 can adequately deliver peak hour demands to most of the Zone under the City's minimum pressure criteria of 40 psi. These locations with pressures below 40 psi are within 5 psi of the acceptable range so no mitigation is recommended at this time.

During a maximum day plus fire flow demand scenario, results indicate that many areas in the buildout water system in Pressure Zones 1 and 2 could not maintain a minimum system pressure of 20 psi under the required fire flow.

The required upgrades to the buildout system for Zones 1 and 2 are extensive, and completion of pipeline upgrades for the sole purpose of improving fire flow would be cost prohibitive to the existing customers of the City. It is recommended that all 4-inch and 6-inch pipelines constructed before 1960 be replaced to improve these conditions.

ES.7.2 Buildout Plus Dual Interest Areas Water System Evaluation and Recommended Improvements

ES.7.2.1 Water Storage Capacity

The City has sufficient storage to provide demand at buildout plus the addition of Dual Interest Areas A and B.

ES.7.2.2 Pumping Capacity

The buildout + Dual Interest Areas A and B system has a pumping capacity surplus of 875 gpm.

ES.7.2.3 Water Distribution System

During a peak hour demand condition, results indicate that the buildout plus Dual Interest Area system in Pressure Zones 2, 3 and 4 can adequately deliver peak hour demands under the City's minimum pressure criteria of 40 psi.

During a peak hour demand condition, results indicate that the system in Pressure Zone 1 can adequately deliver peak hour demands to most of the Zone under the City's minimum pressure criteria of 40 psi. These locations with pressures below 40 psi are within 5 psi of the acceptable range so no mitigation is recommended at this time.

During a maximum day plus fire flow demand scenario, results indicate that many areas in the buildout plus Dual Interest Area water system in Pressure Zones 1 and 2 could not maintain a minimum system pressure of 20 psi under the required fire flow.

The required upgrades to the buildout plus Dual Interest Area system for Zones 1 and 2 are extensive, and completion of pipeline upgrades for the sole purpose of improving fire flow would be cost prohibitive to the existing customers of the City. It is recommended that all 4-inch and 6-inch pipelines constructed before 1960 be replaced to improve these conditions.

Because Dual Interest Areas A and B will be annexed into Pressure Zone 2, the following recommended improvements are required for the future system in Pressure Zone 2.

- Installation of approximately 6,060 linear feet of 8-inch diameter ductile iron (DI) pipeline to provide backbone infrastructure to this new area.
- Installation of approximately 4,570 linear feet of 8-inch diameter DI pipeline to provide backbone infrastructure to this new area.

ES.7.3 UGMA Water System Evaluation

ES.7.3.1 Water Storage Capacity

The addition of the UGMA, excluding the Dual Interest Areas, to the City's water system would increase water demand by approximately 4.5 mgd; nearly tripling the City's current water demand. Because of this large increase, and because the actual timing of developments and annexations will vary as political, environmental or other conditions develop, specific recommendations for UGMA future storage are beyond the scope of this Water System Master Plan. The City's future water storage capacity could be increased by the construction of additional storage facilities as well as the addition of new groundwater wells which could provide an increase in the City's available groundwater storage credit.

ES.7.3.2 Pumping Capacity

The City has a pumping capacity deficiency of 4,631 gpm for the UGMA Water Distribution System. The City's future groundwater pumping capacity could be increased by the construction of additional groundwater wells as well as increasing water rights.

ES.7.4 Recommended Capital Improvement Program

A detailed description of the City's CIP is provided in Chapter 9. Recommendations for improvements to the existing and future water system are described in Chapters 7 and 8, respectively. The following subsections present a brief overview of the recommended CIP for the City.

ES.7.4.1 Existing System Improvements

Chapter 7 provided a description of the evaluation of the City's existing water system and its ability to meet the established operational and design criteria described in Chapter 5. Based on the evaluation, several improvements to the existing system were recommended to eliminate existing deficiencies, as listed in Table ES-3 and illustrated on Figure ES-3. A summary of the recommended capital improvements to the existing system is listed below.

Table ES-3. Recommended Pipeline CIP for Existing System

CIP ID	Pressure Zone	Description of Location	Length, feet	Diameter, inches	
				Existing	Recommended
PH01	1	Reconfigure Southwest portion of Zone 1 Boundary	450	-	8
Public Area Fire Flow Improvements					
FF01	1	From hydrant to Willard Street	320	6	8
FF02	2	Area west of Flavel Drive	600	6	8
Fire Flow Improvements: Pipelines Constructed Prior to 1960					
FF03	1	See Figure 9-1	10	4	8
FF03	2	See Figure 9-1	10,582	4	8
FF03	3	See Figure 9-1	2,975	4	8
FF03	1	See Figure 9-1	15,156	6	8
FF03	2	See Figure 9-1	49,373	6	8
FF03	3	See Figure 9-1	5,329	6	8
FF03	4	See Figure 9-1	361	6	8

PH: Indicates a project to resolve peak hour deficiencies.
FF: indicates a project to resolve fire flow deficiencies.

ES.7.4.1.1 *Water Storage Improvements*

- Install a remote controlled shut-off valve or seismic valve at the Elevated Reservoir.
- Install a remote controlled shut-off valve or seismic valve at the Concrete Reservoir.
- Install a remote controlled shut-off valve or seismic valve at the Stanley Reservoir.

ES.7.4.1.2 *Water Pumping Improvements*

- Install two additional 1,750 gpm fire flow pumps to the Third Pressure Zone Booster Pump Station.

ES.7.4.1.3 *Water System Facility Maintenance*

- Prepare and recoat the exterior of the Stanley Tank.
- Prepare and recoat the top of the exterior of the Elevated Tank.
- Perform periodic well maintenance, including well pump removal and rehabilitation in Pressure Zone 4.

ES.7.5 Future System Improvements

Chapter 8 provides a description of the evaluation of the City's future water system and its ability to meet the established operational and design criteria described in Chapter 5. Based on the evaluation, several improvements to the future system were recommended to eliminate future deficiencies, as listed in Table ES-4 and illustrated on Figure ES-4. These have been grouped into recommended Buildout CIP (BCIP) projects and are listed below by pressure zone.

Table ES-4. Recommended Pipelines CIP for Buildout System

CIP ID	Pressure Zone	Description	Length, feet	Diameter, inches	
				Existing	Recommended
BDIA01	2	Infrastructure to support Dual Interest Area A	6,060	NA	8
BDIA02	2	Infrastructure to support for Dual Interest Area B	4,570	NA	8

ES.7.5.1 Water Storage Improvements

- Perform periodic well maintenance, including well pump removal and rehabilitation.

ES.7.6 Recommended Cost and Timing of Capital Improvements

Costs are presented in January 2011 dollars based on an Engineering News Record Construction Cost Index (ENR CCI) of 8,938 (20 Cities Average). Total CIP costs include the following construction contingency and project cost allowances:

- Construction Contingency: 20 percent
- Project Cost Allowances:
  - Design: 10 percent
  - Construction Management: 10 percent
  - Administration: 8 percent

A summary of the costs of the recommended CIP by project type is provided in Table ES-5. As shown in Table ES-5, the total estimated recommended CIP cost for the City system is $23.18 million.

Table ES-5. Estimated Cost of Recommended CIP by Project Type

CIP Project Type	Existing System CIP Projects	Buildout System CIP Projects
Pipelines	19.27	2.41
Storage Facility Maintenance	0.36	-
Water Storage Facility Improvements	0.07	
Pump Stations	0.77	-
Emergency Generators	-	-
Pressure Reducing Stations	-	-
Groundwater Well Maintenance	0.08	0.23
Total CIP Cost	$20.54 million	$2.64 million

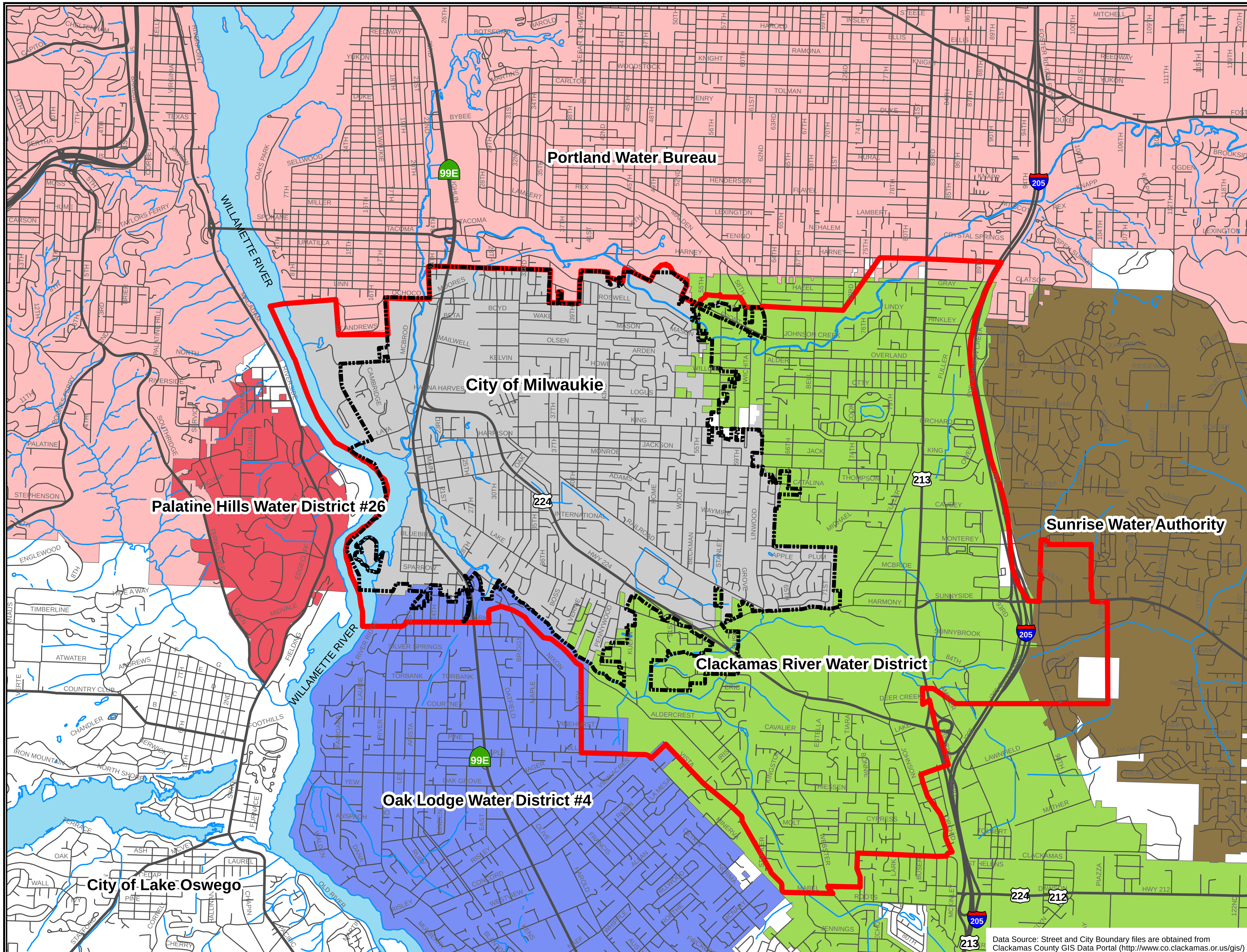
The recommended improvements for the existing system should be completed within the next five years.

The construction of the improvements for the future system should be coordinated with the proposed schedules of future development to ensure that the required infrastructure will be in place to serve future customers.

FIGURE ES-1

City of Milwaukie
2010 Water System Master Plan

2010 SERVICE AREA AND NEIGHBORING WATER SUPPLIERS



N

0 1,350 2,700
Scale in Feet

LEGEND

- City Boundary
- Milwaukie Service Area
- Urban Growth Boundary
- Freeways & Highways
- Rivers & Streams

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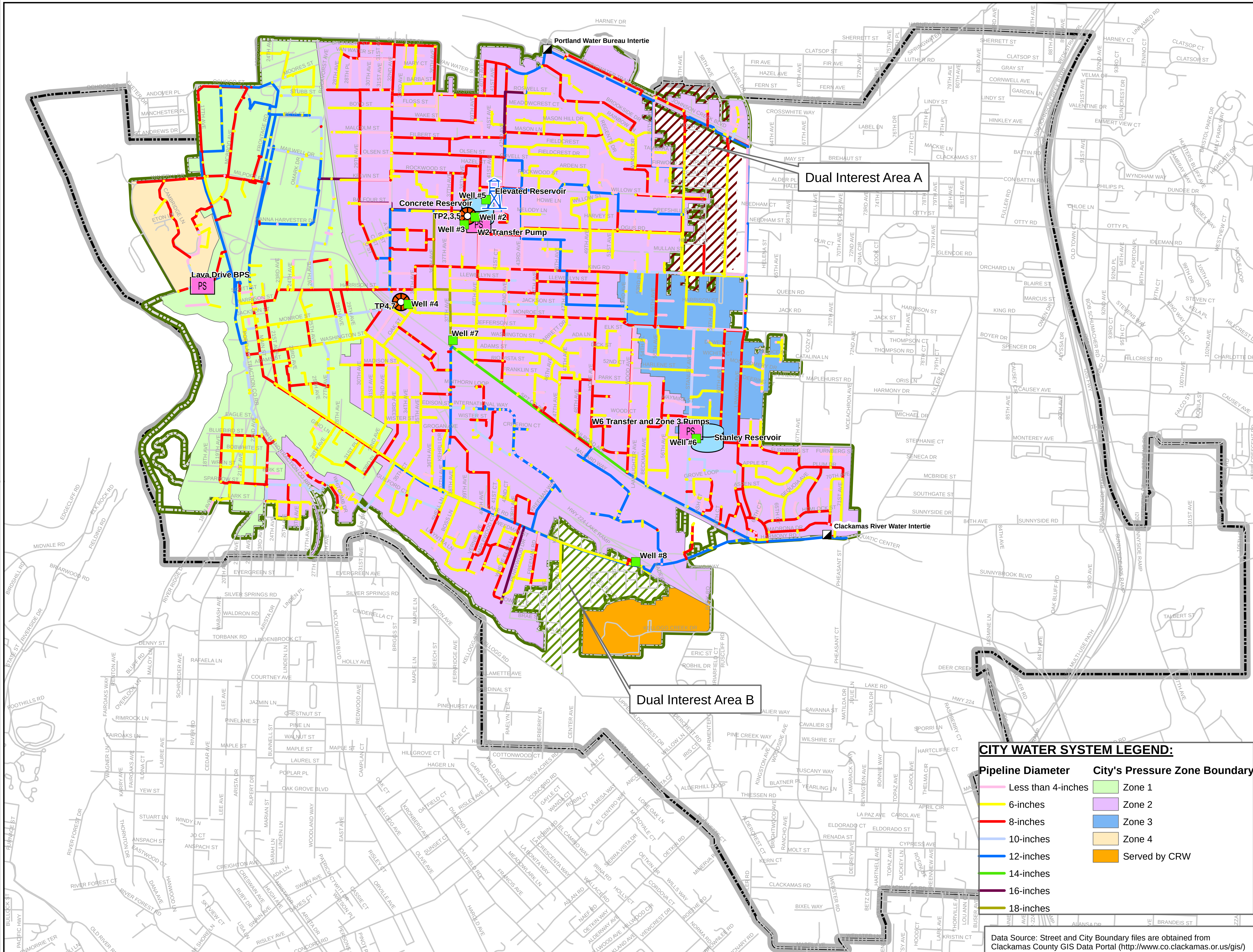
FIGURE ES-2

City of Milwaukie
2010 Water System Master Plan

2010 WATER SYSTEM



N

0 1,050 2,100

Scale in Feet

LEGEND

- PS Pump Station
- At Grade Reservoir
- Elevated Reservoir
- Treatment Tower
- Well
- Emergency Interties
- Dual Interest Area B
- Dual Interest Area A
- City Boundary
- Urban Growth Boundary

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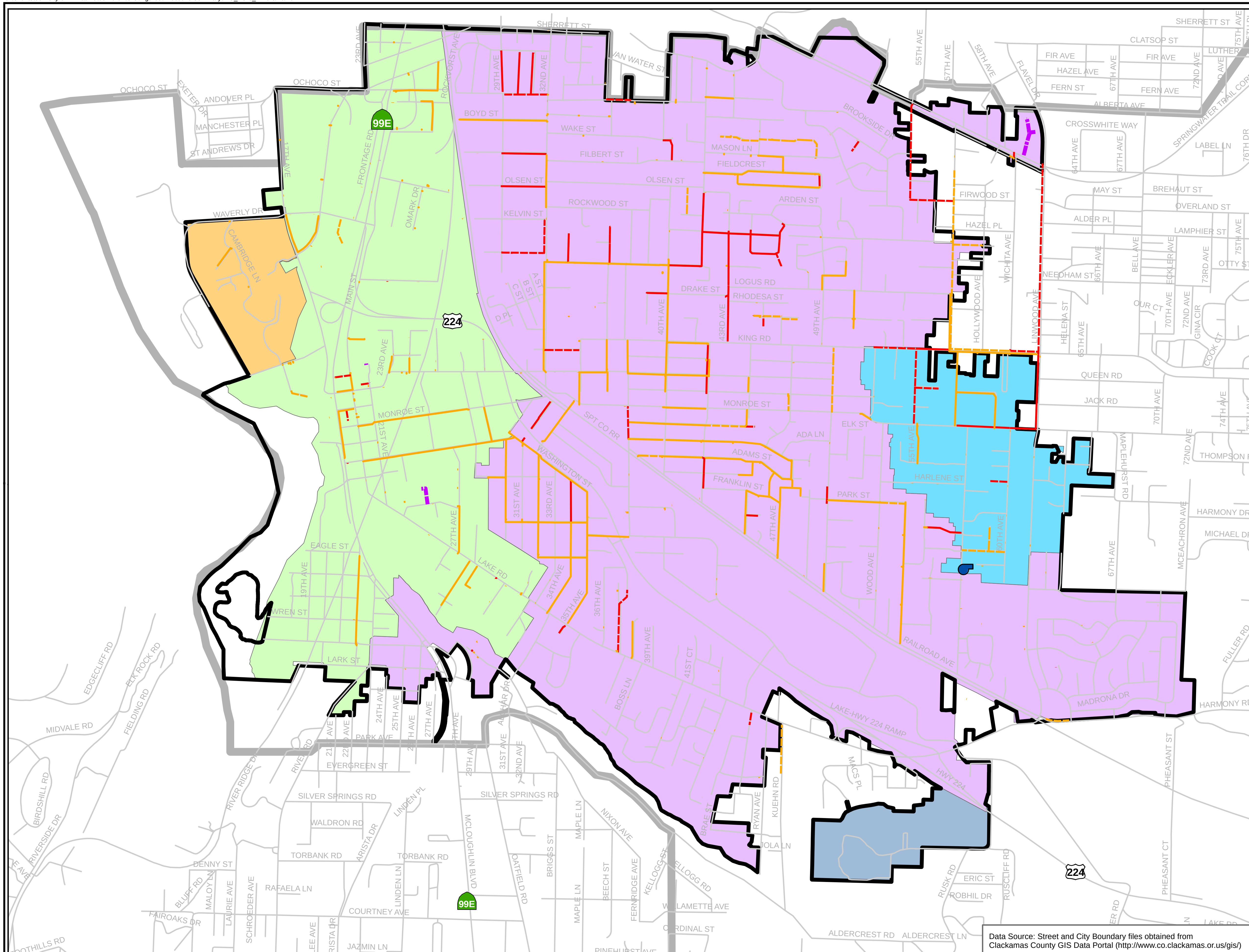

FIGURE ES-3

City of Milwaukie
2010 Water System Master Plan

EXISTING SYSTEM IMPROVEMENTS

N

0 750 1,500

Scale in Feet

LEGEND

- Public Area Fire Flow Improvements (FF01-FF02)

CIP Pipeline Improvements (FF03)
- 4-inch Diameter, Pre-1960
- 6-inch Diameter, Pre-1960
- 4-inch Diameter, Year Unknown
- 6-inch Diameter, Year Unknown
- Zone 3 Fire Flow Pumps

City's Pressure Zone Boundary
- Zone 1
- Zone 2
- Zone 3
- Zone 4
- Clackamas River Water
- City Boundary
- Urban Growth Boundary
- Street

Data Source: Street and City Boundary files obtained from Clackamas County GIS Data Portal (http://www.co.clackamas.or.us/gis/)

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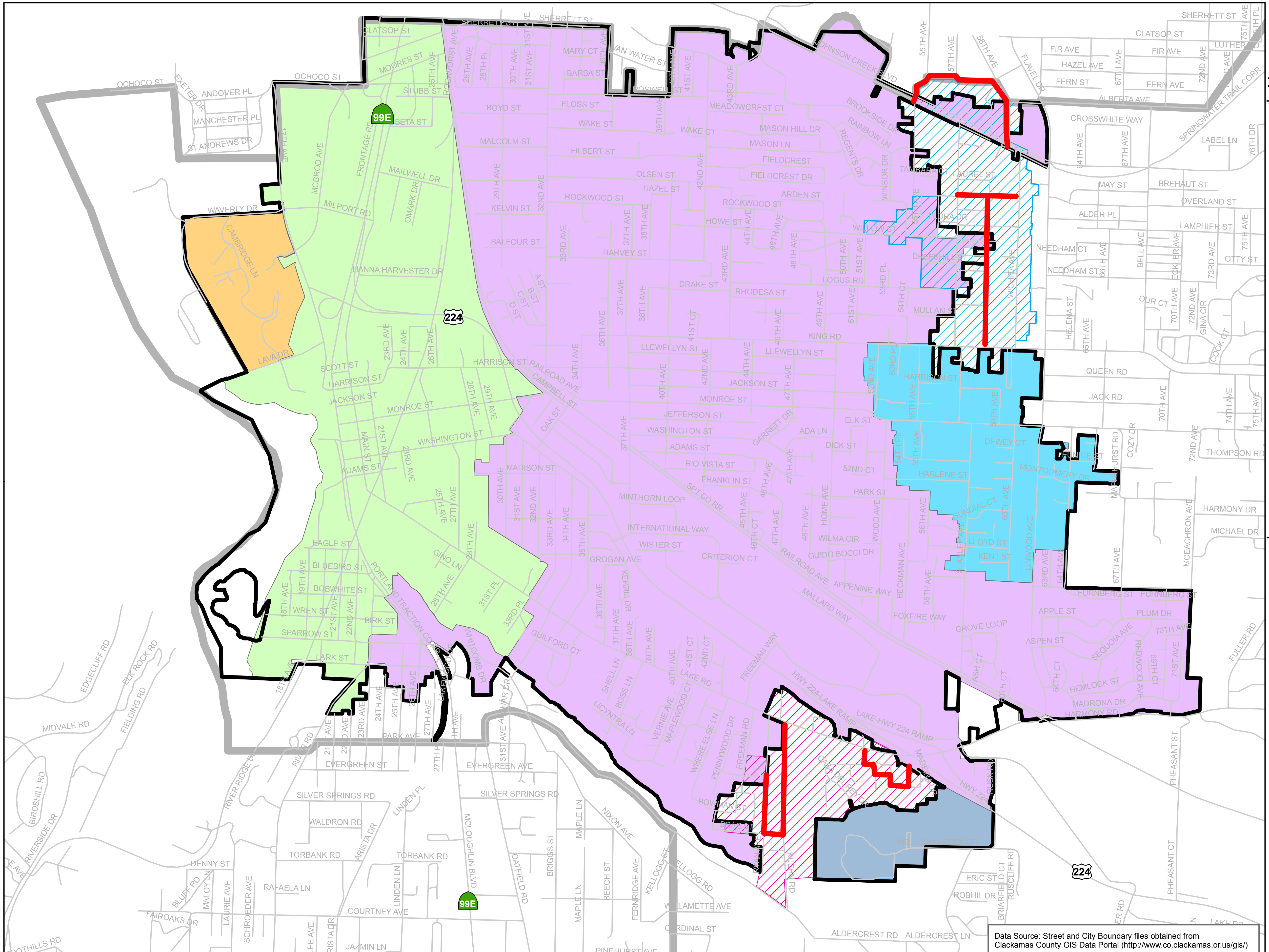

FIGURE ES-4

City of Milwaukie
2010 Water System Master Plan

BUILDOUT SYSTEM IMPROVEMENTS

N

0 750 1,500
Scale in Feet

LEGEND

City's Pressure Zone Boundary

- Zone 1
- Zone 2
- Zone 3
- Zone 4
- Clackamas River Water
- City Boundary
- Dual Interest Area A
- Dual Interest Area B
- Urban Growth Boundary
- Street
- 8-inch Diameter DI Pipeline

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CHAPTER 1
Introduction

1.1 2010 WATER SYSTEM MASTER PLAN PURPOSE

This 2010 Water System Master Plan for the City of Milwaukie (City) identifies strategies for maintaining adequate water supplies and service levels for the community; guides capital expenditures for the system; furnishes important guidance on operational issues; and charts a course for future updates to water rates. To accomplish these goals, the following work tasks were performed in the Water System Master Plan:

- Evaluate and summarize existing water system and key system facilities;
- Develop water demand projections through buildout;
- Evaluate existing and future water supplies to develop a strategy for the City to meet existing and future water demands;
- Develop performance and operational criteria under which the water system will be analyzed and future facilities will be formulated;
- Develop and calibrate a water distribution system hydraulic model;
- Evaluate existing and buildout water system conditions to identify the City's water distribution system facility needs; and,
- Develop a capital improvement program for recommended existing and future water system facilities.

1.2 AUTHORIZATION

West Yost Associates (West Yost) was authorized to prepare this 2010 Water System Master Plan by the City on February 5, 2010.

1.3 REPORT ORGANIZATION

This Water System Master Plan is organized into the following chapters:

Chapter 1: Introduction
Chapter 2: Service Area Characteristics
Chapter 3: Existing Water System
Chapter 4: Water Demand
Chapter 5: Water Distribution System Service Standards
Chapter 6: Hydraulic Model Development
Chapter 7: Evaluation of Existing Water System
Chapter 8: Evaluation of Future Water System
Chapter 9: Recommended Capital Improvement Program

The following appendices to this Water System Master Plan contain additional technical information, assumptions and calculations:

Appendix A: Hydraulic Model Calibration – Hydrant Tests

Appendix B: HPR Locations and Verification Results

Appendix C: Cost Estimating Assumptions

1.4 ACRONYMS AND ABBREVIATIONS

The following acronyms and abbreviations have been used throughout this Water System Master Plan to improve document clarity and readability.

ADD	Average Day
AWWA	American Water Works Association
BCIP	Buildout CIP
BP	Business Park
C	Commercial
C/HD	Mixed Use
C2	Community Commercial
C3	General Commercial
CC	Corridor Commercial
CCFD	Clackamas County Fire District #1
CCSD	Clackamas County Service District No. 1
C-factor	Pipeline Friction Factor
CIP	Capital Improvement Plan
City	City of Milwaukie
CRW	Clackamas River Water District
D/DBPR	Disinfection/Disinfection By-Product Rule
DBPs	Disinfection Byproducts
DHS	Oregon State Department of Human Services
ECAC	Engineering Computer Applications Committee
ENR CCI	Engineering News Record Construction Cost Index
EPA	Environmental Protection Agency
EPS	Extended Period Simulation
fps	Feet Per Second

ft/kft	Feet Per Thousand Feet
gpcd	Gallons Per Capita Per Day
gpm	Gallons Per Minute
HAA5	5 Major Haloacetic Acids
HD	High Density
HDR	High Density Residential
HPR	Hydrant pressure recorder
I or M	Industrial
I2	Light Industrial
I3	General Industrial
IDSE	Initial Distribution System Evaluation
ISO	Insurance Services Office, Inc.
LD	Low Density
LIDAR	Light Detection and Ranging
LRAA	locational running annual average
LTIC	Low Traffic Impact Commercial
MCLs	Maximum Contaminant Levels
MD	Moderate Density
MDD	Maximum Day Demand
MED.D	Medium Density
MG	Million Gallons
mgd	Million Gallons Per Day
MR1	Medium Density Residential
MR2	Medium High Density Residential
NFPA	National Fire Protection Association
OAR	Oregon Administrative Rules
OC	Office Commercial
OFC	Oregon Fire Code
OLWD	Oak Lodge Water District
OSM	Open Space Management
PHD	Average Day to Peak Hour Demand
PRV	Pressure Reducing Valve

R7 or R10	Urban Low Density Residential
RCC	Regional Center Commercial
RCHD	Regional Center High Density
RCO	Regional Center Office
RTL	Retail Commercial
SCADA	Supervisory Control and Data acquisition
SDWA	Safe Drinking Water Act
SMP	Standard Monitoring Plan
Stage 2 DBPR	Stage 2 Disinfectants and Disinfection Byproducts Rule
SWA	Sunrise Water Authority
SWTR	Surface Water Treatment Rule
TC	Town Center
TCR	Total Coliform Rule
THM	Total Trihalomethanes
TP235	Water Treatment Plant 235
TP47	Water Treatment Plant 47
UAFW	unaccounted-for water
UGB	Urban Growth Boundary
UGMA	Urban Growth Management Area
UGMA	Urban Growth Management Area
VFD	Variable Frequency Drive
VOCs	Volatile Organic Compounds
West Yost	West Yost Associates

1.5 ACKNOWLEDGMENTS

The development of this Water System Master Plan would not have been possible without the key involvement and assistance of City staff. In particular, the following staff provided comprehensive information, significant input and important insights throughout the development of this Water System Master Plan:

- Zachary Weigel P.E., Project Manager
- Jason Rice P.E., Assistant Project Manager
- Gary Parkin P.E., Engineering Director
- Ryan Marquardt A.I.C.P., Associate Planner
- Dave Butcher, Asset Management Technician
- Mike Clark, Water Operations Department Manager
- Don Simenson, Water Quality Coordinator
- Jamie Clark, Water Utility Lead

CHAPTER 2
Service Area Characteristics

The purpose of this chapter is to describe the City's existing (2010) service area characteristics. System information has been obtained through the review of previous reports, maps, plans, operating records, interviews and other available data provided by the City. The following sections of this chapter describe the components of the City's service area:

- Service Area
  - Existing Service Area
  - Land Use
  - Urban Growth Management Areas
  - Dual Interest Areas
  - Hydrology and Water Sources
  - Topography
- Population Served

2.1 SERVICE AREA

This section describes the existing service area by its geographical features and its land use. This section also discusses growth areas that are currently unserved and those served by other water providers.

2.1.1 Existing Service Area

The City currently provides potable water service to most areas within the City limits, though some residents are served by other providers. The City is located in the Portland Metropolitan Area approximately 7 miles south of downtown Portland, and is bounded on the west by the Willamette River, the north roughly by Johnson Creek Boulevard, the east roughly by Linwood Avenue and 71st Avenue, and the south by Kellogg Creek and Lake Road. The City limits and service area are shown in Figure 2-1, and include approximately 3,169 acres, or about 4.95 square miles.

Also shown on Figure 2-1 are neighboring water purveyors that include the City of Portland to the north, Oak Lodge Water District (OLWD) to the south and the Clackamas River Water District (CRW) and Sunrise Water Authority (SWA) to the east. The City of Portland serves approximately 163,000 retail customers and covers 143 square miles in their retail service area bordering three counties. Adjacent to the Willamette River, the OLWD was formed in 1922 and has a retail service population of approximately 28,000 customers. CRW serves areas that are mostly in unincorporated Clackamas County east of the Willamette River. Their customer base is approximately 80,000 customers including retail and wholesale. The SWA serves an area of approximately 22 square miles and encompasses the communities of Happy Valley and Damascus, as well as, unincorporated county areas with a total service population of about 40,000.

2.1.2 Land Use

The City has a mix of users including various density residential areas, commercial, industrial and public. The City also has a town center designation in the old downtown area near the Willamette River. Current comprehensive plan land use designations for the City are shown on Figure 2-2.

As shown on Figure 2-2, the City is approximately 97 percent developed and there are few vacant areas within the City limit. Table 2-1 summarizes the acreage of the vacant parcels by land use category. Outside the Town Center, about 68 acres remain available for residential construction. There are approximately 12 acres of land zoned industrial available for development.

Table 2-1. Vacant Parcels Within City Limits

City Land Use Category	Acres[a]
Commercial (C)	0.1
Mixed Use (C/HD)	2.0
High Density (HD)	6.2
Industrial (I)	11.9
Low Density (LD)	41.0
Moderate Density (MD)	6.9
Medium Density (MED.D)	13.9
Town Center (TC)	8.4
Total Vacant Acres	90.4
Total City Acres[b]	3,085.0

[a] Based on City parcel data provided by the City in Item 001 - Milwaukie Geodatabase.
[b] Total acreage is based on land use map provided by the City in Item 004 - Comp Plan and Maps. Total acreage does not include right of way and roads.

2.1.3 Urban Growth Management Areas

The City is under the governance of an elected, regional governing body called Metro. In 1990, approximately 7,400 acres of land adjacent to the City was designated as an Urban Growth Management Area (UGMA). This means that these lands will be the first areas of growth for the region. Generally, the UGMA surrounds the City with the majority of the growth management area located to the east and southeast of the City limits, as shown on Figure 2-3. Being that these lands are adjacent to the City, some, or all, of the areas could be annexed into the City.

Although most of these UGMA lands are located within unincorporated Clackamas County, they are already highly developed. Based on the vacant land inventory, eleven percent of the area is currently vacant. Table 2-2 summarizes the vacant area by land use.

Table 2-2 Urban Growth Management Area Vacant Land

County Land Use Category	Acres
Business Park (BP)	12
Community Commercial (C2)	1
General Commercial (C3)	5
Corridor Commercial (CC)	5
High Density Residential (HDR)	7
Light Industrial (I2)	32
General Industrial (I3)	22
Low Traffic Impact Commercial (LTIC)	1
Medium Density Residential (MR1)	3
Medium High Density Residential (MR2)	1
Office Commercial (OC)	31
Open Space Management (OSM)	107
Urban Low Density Residential	148
Regional Center Commercial (RCC)	3
Regional Center High Density (RCHD)	5
Regional Center Office (RCO)	4
Retail Commercial (RTL)	2
Outside County Zoning Area	5
UGMA Vacant Land Total	394
UGMA Land Total	3,705
Percentage of Vacant Land	11%

Also, the lands within the UGMA receive water service from special districts and the City of Portland. The largest portion of the UGMA is served by the Clackamas River Water District (CRW), while a small area east of Interstate 205 is served by the SWA, an area south of downtown is served by the OLWD, and a very small area to the north is served by the City of Portland.

It is unknown at this time how much of the UGMA will annex into the City, or when such annexations will occur. The fact that the UGMA lands are already developed and receive full utility service provides less incentive to annex into the City. However, the possibility that these lands could annex into the City at some point in the future creates a need for the City to have a clear understanding of the potential impact that annexation would have on the water system. Such an understanding will help guide policy decisions regarding future annexation into the City and the supply of water service.

2.1.4 Dual Interest Areas

Within the UGMAs lies a subset of land known as Dual Interest Areas, shown on Figure 2-3. These areas are almost entirely surrounded by the City, but currently receive water service from CRW. These are areas that may come into the City's service area in the near future and therefore need careful planning and consideration.

Dual Interest Area A is in the northeast corner of the City and is bounded by Johnson Creek Boulevard on the north, Wichita Avenue to the East and King Road on the south. This area currently receives water service from CRW. The requirement to annex into the City and connect to recently constructed sewers has increased the likelihood the City will provide water service to this area in the future. As a result, the City must ensure the City water system is prepared and capable of extending water service to this area.

Dual Interest Area B is in the southeast corner of the City bounded by Highway 224 on the north and intersected by Kuehn Road. Customers in this area receive water service from CRW. Clackamas County Service District No. 1 provides sewer service in this area. Like the UGMA lands, this area is highly developed and currently receives full services. As a result, there is less incentive to annex into the City, and it may be some time before privately driven annexations are sought. However, this area is nearly surrounded by the City limits. A few key developments and annexations could create an island and annexation of the area could be forced. Such a situation necessitates the City be prepared to provide water service should annexation occur.

Both Dual Interest Areas A and B have a small amount of vacant land which is summarized in Table 2-3.

Table 2-3 Dual Interest Area Vacant Land

County Land Use Category	Acres
Dual Interest Area A	
General Industrial (I3)	1
(M)Industrial	1
Urban Low Density Residential (R7)	5
Urban Low Density Residential (R10)	2
Dual Interest Area A Vacant Land Total	9
Dual Interest Area A Land Total	140
Dual Interest Areas A Percentage of Vacant Land	6%
Dual Interest Area B	
Urban Low Density Residential (R7)	1
Urban Low Density Residential (R10)	17
Dual Interest Area B Vacant Land Total	18
Dual Interest Area B Land Total	97
Dual Interest Area B Percentage of Vacant Land	19%

2.1.5 Hydrology and Water Sources

The City is bounded on the west by the Willamette River, whereas Johnson Creek traverses the northern area and Kellogg Creek traverses the southern area of the City. The Clackamas River runs east to west just three miles south of the City limits. While both rivers are used as sources for drinking water in the Portland Metropolitan area, the City's drinking water is supplied by the underground basin of the Troutdale Aquifer. The Troutdale Aquifer is approximately 300 square miles and is part of the larger Portland Basin that includes portions of the States of Oregon and Washington. The aquifer is synclinal, and the center of the basin is well-confined by low-permeability layers making it a good municipal source.

Whereas the drinking water source for the City comes from an underground basin, the areas surrounding the City are primarily served by surface water. The primary source of drinking water for the City of Portland is the Bull Run Watershed, a protected watershed west of Mount Hood. Additional water for the City of Portland is supplied by groundwater sources at the South Columbia well field, which also taps into the Troutdale Aquifer. CRW, OLWD and the SWA are supplied drinking water from the Clackamas River. SWA also has six wells that are used to meet peak demand.

Supplementing the City's water supply, the City has interconnections with the surrounding water providers, including the City of Portland and the CWD water systems. These additional supplies of water are used only in the event of an emergency.

2.1.6 Topography

The City's service zones range from about 20 feet above sea level to approximately 205 feet of elevation. The rise generally occurs from west to east with the low areas at the Willamette River.

2.2 POPULATION SERVED

According to the Portland State University Population Research Center, the City has a 2009 estimated population of 20,920. Table 2-4 shows population estimates for each year since the last census in 2000. The average annual population growth during the last decade has been 0.20 percent. Since growth in the City is through infill, growth has been much slower than neighboring communities, where land is available for larger scale development.

Table 2-4. City of Milwaukie Historical Population

Year	Population
2000	20,540
2001	20,550
2002	20,550
2003	20,580
2004	20,590
2005	20,655
2006	20,835
2007	20,920
2008	20,915
2009	20,920

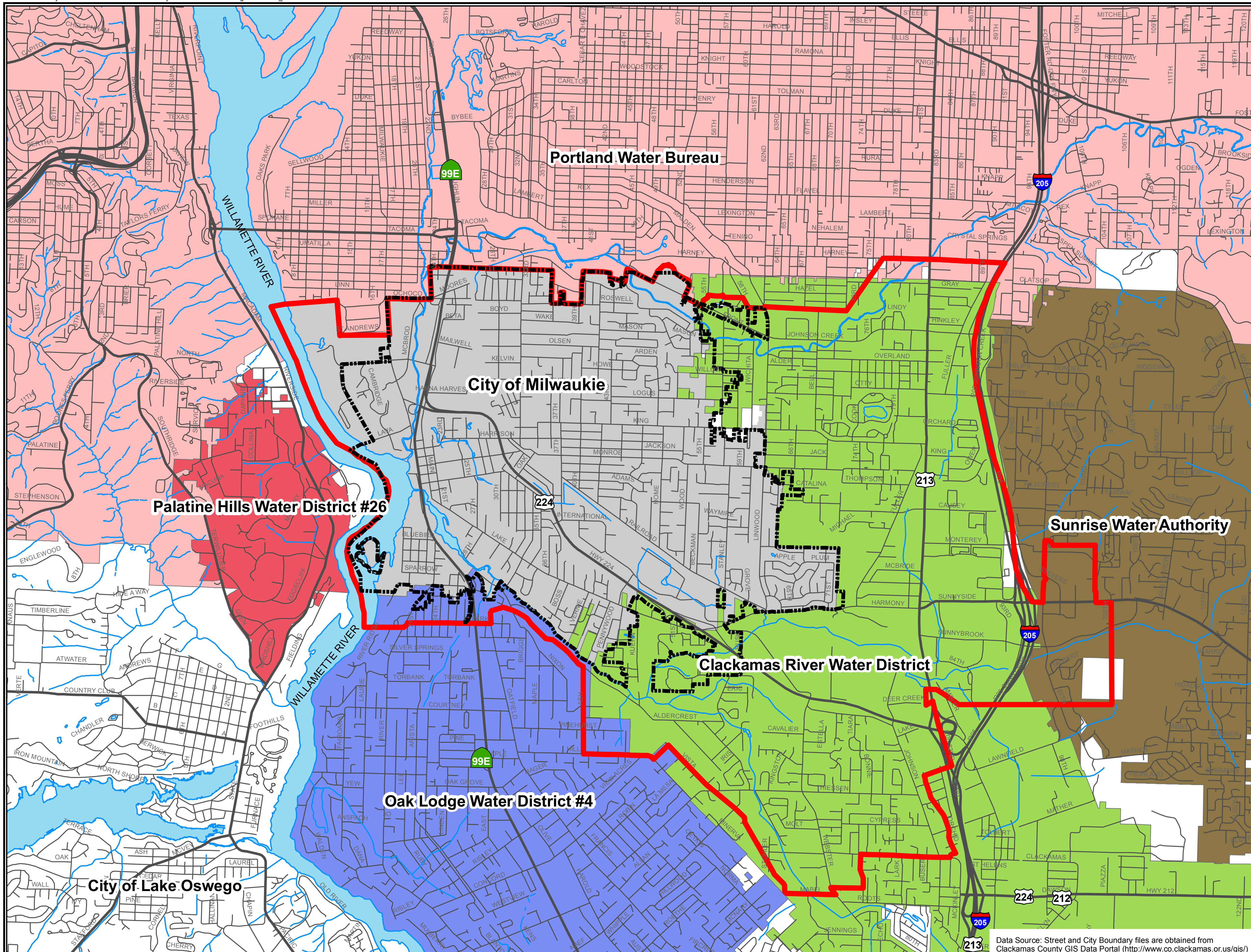
Based on 2010 census data, the population ratio for the City is 2.34 persons per housing unit. Using this ratio, the estimated population of Dual Interest Area A is 930, with approximately 520 people residing outside of the current City limits. The estimated population of Dual Interest Area B is 560, with approximately 530 people residing outside of the current City limits.

P:\Clients\382 City of Milwaukie\03-10-01 2010 Water System Master Plan\GIS\Figures\FIG2-1_Service Area.mxd 8/22/2011

FIGURE 2-1

City of Milwaukie
2010 Water System Master Plan

2010 SERVICE AREA AND NEIGHBORING WATER SUPPLIERS



0 1,350 2,700
Scale in Feet

LEGEND

- City Boundary
- Milwaukie Service Area
- Urban Growth Boundary
- Freeways & Highways
- Rivers & Streams

Data Source: Street and City Boundary files are obtained from Clackamas County GIS Data Portal (http://www.co.clackamas.or.us/gis/)

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P:\Clients\382 City of Milwaukie\03-10-01 2010 Water System Master Plan\GIS\Figures\FIG2-2_Land Use.mxd 11/16/2010

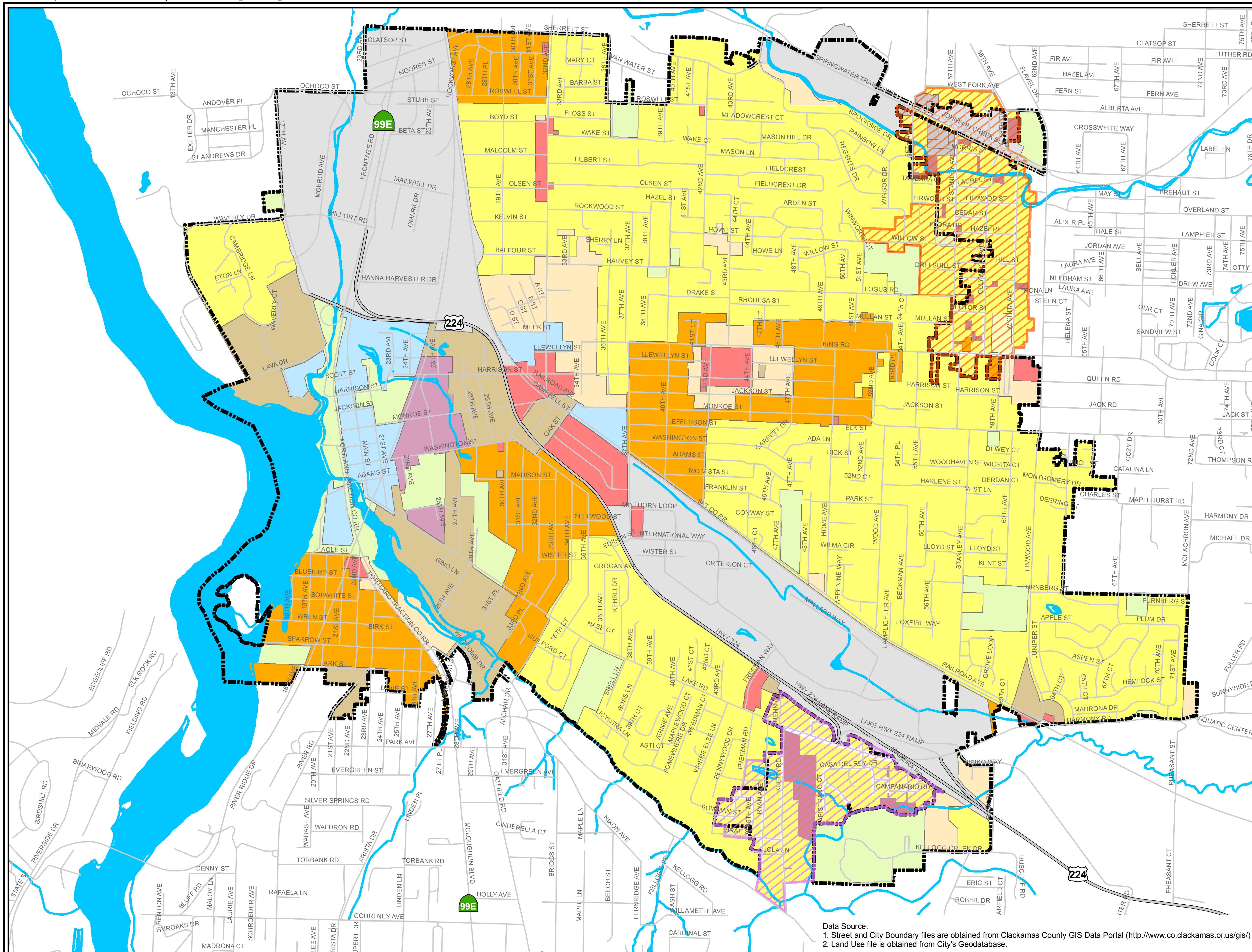

FIGURE 2-2

City of Milwaukie
2010 Water System Master Plan

2010 GENERAL PLAN LAND USE

0 750 1,500
Scale in Feet

LEGEND

Land Use Category
- Low Density Residential
- Medium Density Residential
- Moderate Density Residential
- High Density Residential
- Mixed Use
- Commercial
- Industrial
- Public
- Town Center

Dual Interest Area Land Use
- Commercial
- Industrial
- Multi-Family Residentail
- Rural
- Single Family Residential
- Vacant
- City Boundary
- Dual Interest Area A
- Dual Interest Area B

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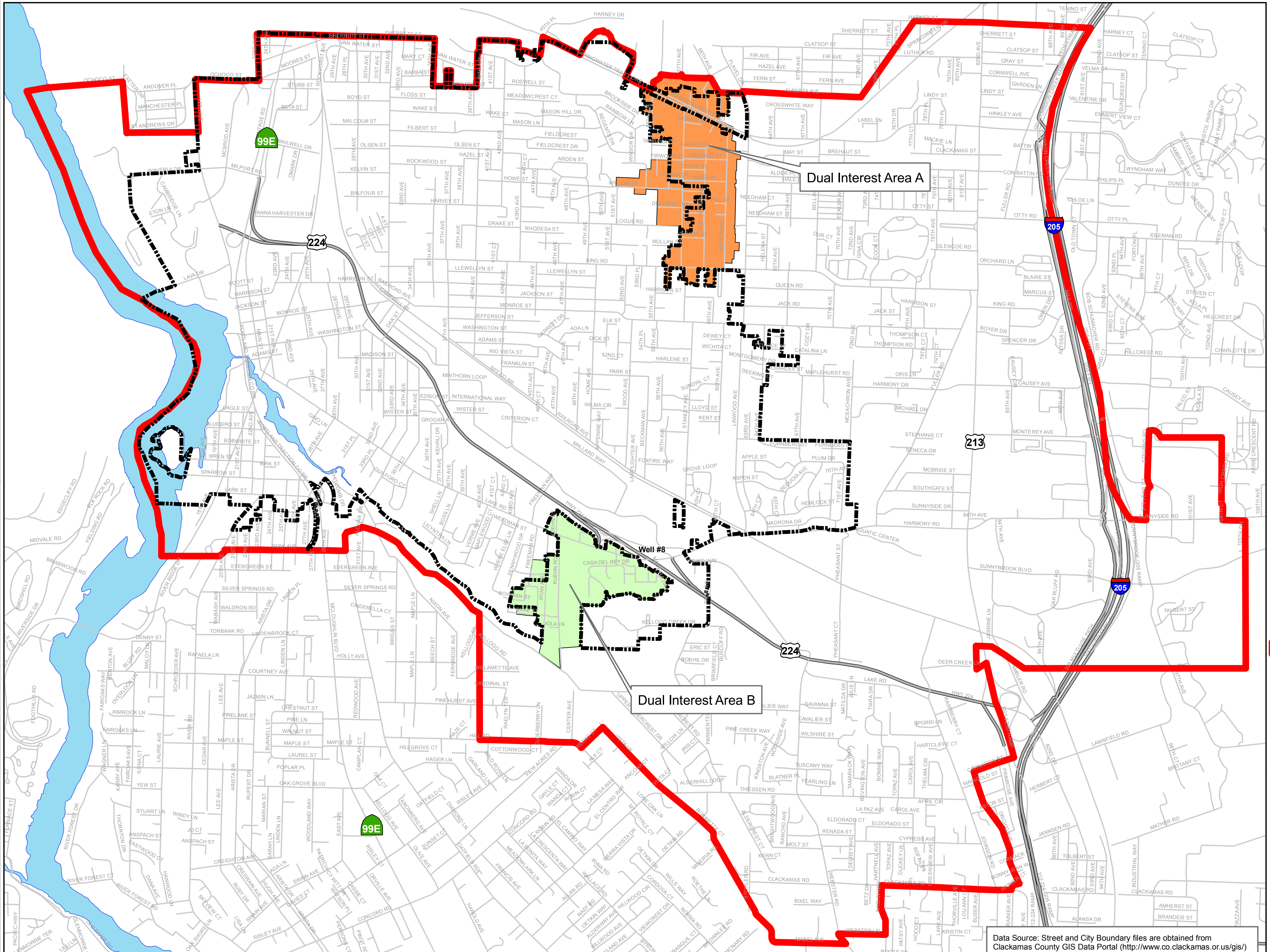
Data Source:
1. Street and City Boundary files are obtained from Clackamas County GIS Data Portal (http://www.co.clackamas.or.us/gis/)
2. Land Use file is obtained from City's Geodatabase.

P:\Clients\382 City of Milwaukie\03-10-01 2010 Water System Master Plan\GIS\Figures\FIG2-3_Growth.mxd 11/29/2010

FIGURE 2-3

City of Milwaukie
2010 Water System Master Plan

2010 URBAN GROWTH AND DUAL INTEREST AREA



0 1,050 2,100
Scale in Feet

LEGEND

- Dual Interest Area A
- Dual Interest Area B
- Urban Growth Boundary
- City Boundary

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CHAPTER 3
Existing Water System

The purpose of this chapter is to describe the City's existing potable water supply and distribution system. System information has been obtained through the review of previous reports, maps, plans, operating records, interviews and other available data provided by the City. The following sections of this chapter describe the components of the City's existing water supply and distribution system:

- Service Connections
- Water Supply
- Water System Facilities
  - — Well Facilities
  - — Water Treatment Facilities
  - — Water Reservoirs
  - — Pumping Stations
  - — Distribution System
  - — Pressure Zones
  - — Telemetry/SCADA System

3.1 SERVICE CONNECTIONS

The City currently has three different revenue classes which make up its 6,787 service connections. A breakdown of the number of connections by revenue class is provided in Table 3-1. The majority of the water system connections are for residential uses, accounting for approximately 93 percent of the total connections to the City water system. An overview of the existing water system is shown in Figure 3-1.

Table 3-1. Existing Number of Service Connections by Revenue Class

Revenue Class	Number of Connections[a]	Percent of Total Connections
Residential	5,971	88
Multiple Density Residential	314	5
Commercial	502	7
Total	6,787	100

[a] Number of connection is based on City of Milwaukie 2009 – 2010 customer billing information.

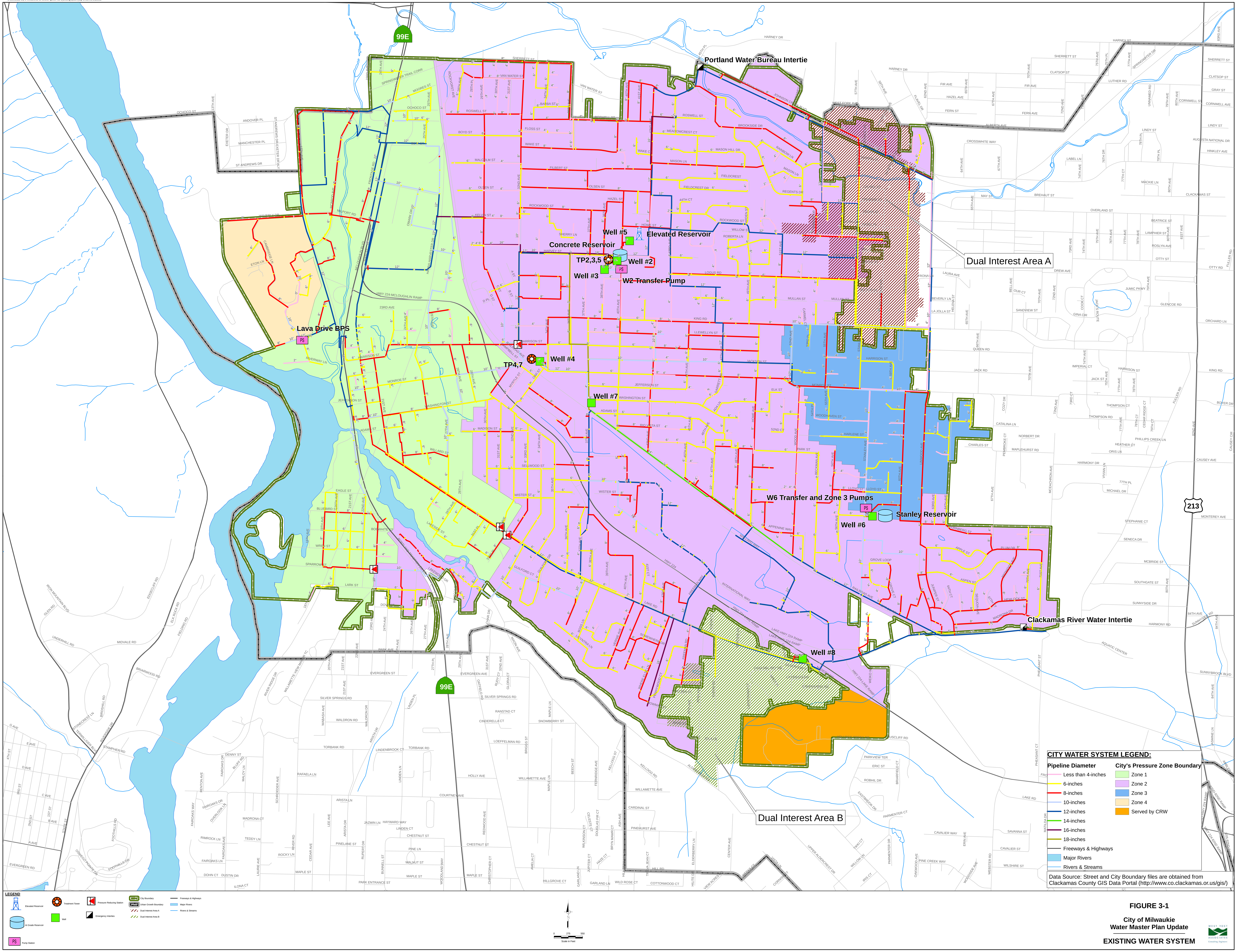

FIGURE 3-1

City of Milwaukie
Water Master Plan Update

EXISTING WATER SYSTEM

3.2 WATER SUPPLY

The City relies entirely on groundwater for its base water supply and has two emergency interties, one with the City of Portland and one with the CRW, both surface water systems. Groundwater from two City-owned wells is pumped directly into the distribution system while water from five other wells is treated before it is pumped into the distribution system. The City's wells pump from the Troutdale Formation that is an extensive aquifer underlying the Portland Metropolitan Area and a large portion of Clark County, Washington. This aquifer is a deep system of gravels and sandstone with large unconsolidated areas. All of the City's wells have active water rights that are certified through the Oregon Water Resources Department. Water rights information for the City's wells is summarized in Table 3-2.

As summarized in the most recent water quality report, water quality for the City surpasses all state and federal standards for drinking water. While Volatile Organic Compounds (VOCs) are present in five of the City's wells, sampling shows that after treatment, the VOCs are not present in delivered water.

Figure 3-2. CRW Intertie Pumping Station

Emergency interties are maintained with the City of Portland, whose primary supply is from the Bull Run system and CRW, whose supply is from the Clackamas River. Both of these interties have bidirectional meters and can operate in either direction.

The CRW intertie is located at 7001 SE Harmony Road and has a pump station in place as shown in Figure 3-2. Pumping capacity for this intertie is approximately 700 gpm in either direction; it pumps into and out of the City Pressure Zone 2.

The Portland intertie, located at Johnson Creek Boulevard and SE 45th Place, is equipped with backflow prevention devices and requires manually controlled bypass pumping for operation to move water from the City to Portland. Moving water from the City of Portland to the City is controlled with a remotely actuated valve, and does not require pumping. The pressure on the City of Portland side is approximately 30 pounds per square inch (psi) higher than the City's Pressure Zone 2.

At one time, the City had an intertie with the OLWD, but this intertie has been disconnected.

Table 3-2. City of Milwaukie Water Rights[a]

Well	Application Number	Permit/Registration Number	Certificate Number	Water Right		
				cfs	gpm	mgd
1 (Inactive)	--	GR-1479	GR-1428	0.85	380	0.5
2	--	GR-1478	GR-1427	0.85	380	0.5
3	--	GR-1480	GR-1429	0.85	380	0.5
4	G-1779	G-1609	G-32158	1.12	503	0.7
5	G-2531	G-2542	G-34010	1.6	718	1.0
6	G-10760	G-9953	G-56403	1.80	808	1.2
7	G-10762	G-9954	G-56404	2.67	1,198	1.7
8	G-11464	G-10582	G-82571	1.62	727	1.0
Total Water Rights				11.36	5,094	7.3

[a] Data collected from Oregon State Water Resources Department records of applications, permits and certificates of water rights.

3.3 WATER SYSTEM FACILITIES

The City's existing system facilities consist of wells, treatment facilities, storage reservoirs, pump stations and pressure reducing valves (PRVs). With their locations shown on Figure 3-1, these facilities are described below, while the evaluation of facility capacities and their ability to meet existing and future potable water demands is described in *Chapter 7, Evaluation of Existing Water System,* and *Chapter 8, Evaluation of Future Water System,* respectively.

3.3.1 Well Facilities

The City has eight wells of which seven are operational. Well No. 1 is off line with capacity used by Wells 2, 3 and 5. Wells 2 through 8 have a combined permitted production capacity of 5,094 gallons per minute (gpm) or 7.3 million gallons per day (mgd). Water from Wells 2, 3, 4, 5 and 7 have historically contained elevated VOCs which is removed using packed tower aeration treatment. These treatment facilities are described in the next section. Wells 2, 3, and 5 are located close to each other and operate as a single well field, turning on and off together and pumping a total amount of water for the well field as permitted by the State Water Resources Department. Table 3-3 presents a summary of the existing well facilities, their status, and key characteristics.

3.3.1.1 Well No. 2

Well No. 2 is located south of the intersection of SE Harvey Street and SE 40th Avenue, adjacent to the Concrete Storage Reservoir and is part of the Well 2, 3, 5 well field. It pumps approximately 394 gpm directly into Tower No. 2 at the Water Treatment Plant 235 (TP235). The on/off operation of Well No. 2 is controlled by the level in the Concrete Storage Reservoir. Well No. 2 pumps into a sand separator and has an on-site back-up generator. Figure 3-3 shows the Well No. 2 pump and sand separator. This is a typical configuration for the City well pumping facilities.

**Figure 3-3. Well No. 2,
Typical Well Discharge Configuration**

Table 3-3. City of Milwaukie Well Facilities

Well Number	Year of Well Construction	Total Depth, feet	Year of Pump Installation	Flow Capacity, gpm[a]	Total Dynamic Head, ft[a]
2	1936	290	1993	394	257
3	1946	290	1980	511	264
4	1960	304	2004	605	290
5	1963	376	1980	605	234
6	1978	336	2007	670	204
7	1984	327	2000	1,120	195
8	2008	481	2009	700	400

[a] Data from System Efficiency Analysis and Recommendations by BacGen.

3.3.1.2 Well No. 3

Well No. 3 is located south of the intersection of SE Harvey Street and SE 40th Avenue, adjacent to the Concrete Storage Reservoir, and is part of the Well 2, 3, 5 well field. It pumps 511 gpm directly into Tower No. 3 at the TP235 site. The on/off operation of Well No. 3 is controlled by the level in the Concrete Storage Reservoir. Well No. 3 has a sand separator in-line with the pump discharge piping and a back-up generator that is located inside the Well No. 2 well house.

3.3.1.3 Well No. 4

Well No. 4 is located at the intersection of SE Monroe Street, SE Railroad Avenue, and SE Oak Street, adjacent to the Water Treatment Plant 47 (TP47). It pumps approximately 605 gpm directly into Tower No. 4 at the TP47 site. The on/off operation of Well No. 4 is controlled by the level in the Elevated Storage Reservoir. Well No. 4 is followed by a sand separator and has an on-site back-up generator.

3.3.1.4 Well No. 5

Well No. 5 is located north of the intersection of SE Harvey Street and SE 40thAvenue, adjacent to the Elevated Storage Reservoir and is part of the Well 2, 3, 5 well field. It pumps approximately 605 gpm directly into Tower No. 5 at the TP235 site. The on/off operation of Well No. 5 is controlled by the level in the Concrete Storage Reservoir. Well No. 5 has an on-site back-up generator and a particle separator that is buried adjacent to the building.

3.3.1.5 Well No. 6

Well No. 6 is located near the intersection of SE Harlow Street and SE Stanley Avenue, adjacent to the Stanley Storage Reservoir. It pumps approximately 670 gpm directly into the Stanley Storage Reservoir. The on/off operation of Well No. 6 is controlled by the level in the Stanley Storage Reservoir. Well No. 6 has an on-site back-up generator, but is not equipped with a sand separator.

3.3.1.6 Well No. 7

Well No. 7 is located near the intersection of SE Washington Street and SE 37th Avenue, a few blocks away from the TP47. It pumps approximately 1,120 gpm directly into Tower No. 7 at the TP47 site. The on/off operation of Well No. 7 is controlled by the level in the Elevated Storage Reservoir. Well No. 7 has a sand separator and an on-site back-up generator.

3.3.1.7 Well No. 8

Well No. 8 is located at 5393 SE Lake Road. It pumps between 300 and 700 gpm directly into the Zone 2 distribution system. The on/off operation of Well No. 8 is controlled by the level in the Elevated Storage Reservoir, and although it has a variable frequency drive (VFD), it is generally operated at a constant speed. Water from Well No. 8 is treated with chlorine which is injected upstream of the chlorine contact chamber that consists of a buried 170 feet. long 72-inch diameter pipe. Well No. 8 also has a sand separator and an on-site back-up generator.

3.3.2 Water Treatment Facilities

Figure 3-4. TP235 Towers

The City operates two treatment facilities that have the same configuration and general operating procedures. Due to VOCs found in Well Nos. 2, 3, 4, 5, and 7, air stripping towers were installed for these wells in 1990. Facility TP235 has three towers and treats the water from Well Nos. 2, 3, and 5, and Facility TP47 has two towers and treats the water from Well Nos. 4 and 7. Water is pumped from the wells directly to its dedicated Tower where air is introduced to strip the VOCs. Chlorine is added to the water for disinfection prior to entering and after leaving the stripping towers. Treated water flows by gravity from the towers to a clearwell below the facility. Vertical turbine booster pumps draw from the clearwell and pump directly into the distribution system for TP47 or to the Concrete Storage Reservoir for TP235. A photo of TP235 is shown in Figure 3-4 and a typical schematic of this system is shown in Figure 3-5. For normal operation, each tower is dedicated to a specific well. Piping is available to change the configuration in the event that one of the towers is not available due to maintenance or other factors.

3.3.3 Water Reservoirs

The City currently operates one elevated steel reservoir (Elevated Reservoir), one ground level steel reservoir (Stanley Reservoir) and one ground level concrete reservoir (Concrete Reservoir) with locations shown in Figure 3-1. Table 3-4 presents a summary of the reservoir type, age, and capacity. As shown in Table 3-4, the City currently has a total above-ground storage capacity of 6.0 million gallons (MG).

Table 3-4. City of Milwaukie Storage Facilities

Storage Facility Name/Number	Storage Type	Material	Year Constructed	Overflow Height, feet	Storage Capacity, MG
Elevated Reservoir	Elevated Tank	Welded Steel	1963	292.4	1.5
Concrete Reservoir	Ground Level	Concrete	1923	211.0	1.5
Stanley Reservoir	Ground Level	Welded Steel	1970	187.3	3.0
				Total Storage Capacity, MG	6.0

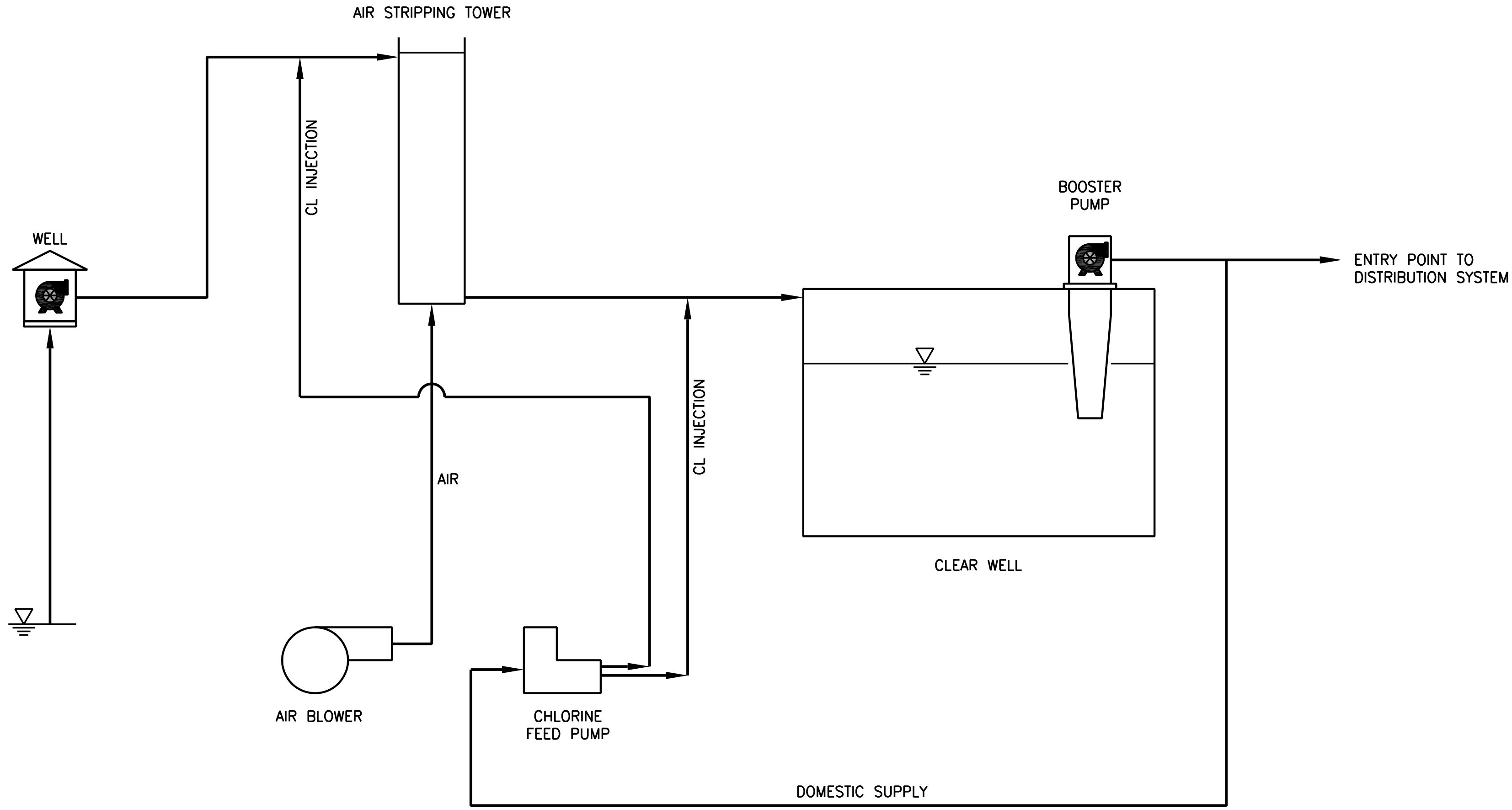

FIGURE 3-5

City Of Milwaukie
2010 Water System Master Plan
TYPICAL VOC TREATMENT SYSTEM SCHEMATIC

WEST YOST ASSOCIATES Consulting Engineers

3.3.3.1 Elevated Reservoir

Constructed in 1963, the Elevated Reservoir was retrofitted for seismic improvements in 2004. This 1.5 MG facility provides storage and gravity supply for the City's Zone 2 portion of the distribution system. It is supplied directly from TP47 and transfer pumps that draw from the City's nearby Concrete Reservoir and via the distribution system from transfer pumps that draw from the City's Stanley Reservoir. The Elevated Reservoir is shown in Figure 3-6.

Figure 3-6. Elevated Reservoir

3.3.3.2 Concrete Reservoir

The concrete reservoir was constructed in 1923, but was suspected of leaking since its construction. In 1995, the reservoir was retrofitted with a liner that operationally appears to have stopped the leaking. This 1.5 MG reservoir is supplied directly by TP235 on Wells 2, 3 and 5. The Concrete Reservoir is the main source of supply for the City's Zone 1 distribution system and the W2 Transfer Pump Station supplies water from the Concrete Reservoir to the Elevated Reservoir via distribution system piping. A photo of the Concrete Reservoir is shown in Figure 3-7.

Figure 3-7. Concrete Reservoir

3.3.3.3 Stanley Reservoir

The Stanley Reservoir is a 3.0 MG at-grade welded steel tank that was constructed in 1970 and is supplied directly from Well No. 6 on the same site. The Stanley Reservoir can also be supplied by Zone 2 distribution piping. This facility is the main source of supply, via post storage booster pumps for the City's distribution system Zone 3. There are also transfer pumps that draw from the Stanley Reservoir and pump into the Zone 2 distribution system and the reservoir could be filled from the Zone 2 distribution system. The Stanley Reservoir is shown in Figure 3-8.

Figure 3-8. Stanley Reservoir

3.3.4 Pumping Stations

The City maintains two transfer pump stations and two booster pump stations. Table 3-5 presents a summary of the existing pumping facilities, their status, and key characteristics. Each pump station configuration is described below.

3.3.4.1 W6 Transfer Pumps

This pumping station is located at the Stanley Reservoir Site inside the Well No. 6 building. It consists of two transfer pumps and two fire pumps that move water from the Stanley Reservoir to Zone 2. The transfer pumps are used regularly to assist in meeting demands in Zone 2 as well as to improve water quality in the Stanley Reservoir. The fire pumps must be manually operated and are rarely used. This pump station has an on-site backup generator with an automatic transfer switch.

Table 3-5. City of Milwaukie Pumping Stations[a]							
Pump Station Name	Pumping From	Pumping To	Number of Pumps	Pump Motor Size and Speed, HP/RPM	Capacity of Each Pump, gpm	Ground Elevation, feet	Rated Discharge Head, feet
W6 Transfer Pumps	Stanley Reservoir	Zone 2	4	50/1750 50/1750 125 125	1300 1300 2250 2250	155	228 228
W2 Transfer Pumps	Concrete Reservoir	Elevated Reservoir	2	20/1800 20/1800	900 900	188	80 80
Lava Drive Booster Pump Station	Zone 1	Zone 4	4	15/3575 15/3575 100/1790 100/1790	300 300 1750 1750	51	116 116 176 176
3rd Pressure Zone Booster Pump Station	Stanley Reservoir	Zone 3	4	15/1800 15/1800 100/3600 100/3600	200 200 600 600	155	108 108 380 380

[a] Data collected from City supplied pump curves and site tour.

3.3.4.2 W2 Transfer Pumps

This pumping station is located inside the Well No. 2 building at the TP235 and Concrete Reservoir site. The two pumps in this station transfer water from the concrete reservoir to the elevated tank to meet the demands of Zone 2 as well as to maintain water quality in the Concrete Reservoir. This station has an on-site back-up generator with a manual transfer switch.

3.3.4.3 Lava Drive Booster Pump Station

The Lava Drive Booster Pump Station provides water to Zone 4 from Zone 1. The two duty pumps at this station supply the normal water demand requirements in Pressure Zone 4. The two fire pumps are activated when the two small pumps cannot maintain the set pressures in the system. This station is a skid mounted station that is housed in a partially buried pre-fabricated vault building as shown in Figure 3-9. This pump station does not have an automatic on-site back-up generator, but has a connection for a trailer mounted generator.

Figure 3-9. Lava Drive Pump Station

3.3.4.4 3rd Pressure Zone Booster Pump Station

The 3rd Pressure Zone Booster Pump Station is located at the Stanley Reservoir site in a building adjacent to the Well No. 6 and W6 Transfer Pump Station building. This station is responsible for meeting the daily and peak demands of Pressure Zone 3 with its two duty pumps and two fire pumps. This station also has a pressure tank, but demands are such that the duty pumps run continuously making the pressure tank obsolete. This station also has an on-site generator that it shares with Well No. 6 and the W6 Transfer Pumps that is automatically activated.

3.3.5 Distribution System

There are approximately 112 miles of pipeline in the City that range in size from 1 to 18 inches in diameter. Table 3-6 provides a summary of pipeline sizes within the service area. Figure 3-1 provides a layout of the City water distribution system.

Table 3-6. City of Milwaukie Pipeline Diameters

Diameter	Length of Pipelines, feet	Percent in Water System
Undefined[a]	2,113	0.36
2	6,600	1.12
4	72,309	12.24
6	201,868	34.17
8	167,419	28.34
10	45,092	7.63
12	78,213	13.24
14	4,792	0.81
16	7,166	1.21
18	4,713	0.80
Total	590,292	100%

(a) There are a total of 88 pipelines (approximately 2,113 feet) without a diameter in the City of Milwaukie geodatabase.

3.3.6 Pressure Zones

The City water distribution system has four pressure zones numbered sequentially. Zones 1 and 2 are fed by gravity from storage reservoirs and range in elevation from 28 to 125 feet and 50 to 195 feet, respectively. Zones 3 and 4 are both fed from pumping stations. Zone 3 ranges from 160 to 205 feet in elevation and Zone 4 ranges from 75 to 150 feet in elevation. This layout is shown schematically on the hydraulic profile in Figure 3-10.

The City operates several pressure reducing stations to manage water pressure between zones, Table 3-7 summarizes the existing stations. The first two stations can be used in the event of low pressures in Zone 4, and to help circulate a small amount of water through Zone 4. The other four stations are used to move water from Zone 2 to Zone 1 when the pressure in Zone 1 drops below the desired pressure.

3.3.7 Telemetry/SCADA System

The City has a complete Supervisory Control and Data Acquisition (SCADA) system that monitors all facilities in the supply and distribution systems. This system includes remote operation and monitoring of facilities, and is controlled at the City's Johnson Creek Facility at 6101 SE Johnson Creek Boulevard.

Table 3-7. Pressure Reducing Stations

Station	Street	Cross Street	From Zone	To Zone	PRV Setting or Control Used in Hydraulic Model, psi	Diameter, in	PRV Elevation in Hydraulic Model	Notes
V-PRV-1	SE Waverly	17th	1 4	4 1	Opens on lower Zone 4 pressure. Open.	8 2	92 92	Operates as a check valve. Set to pass about 20 gpm.
V-PRV-2	SE McBrod	17th	1	4	Opens on lower Zone 4 pressure.	8	110	Operates as a check valve.
V-PRV-3	Harrison	32nd	2	1	43	8	102	Opens on Zone 1 pressure lower than Elev. 202 in Concrete Reservoir.[a]
V-PRV-4	Lake	33rd	2	1	40	8	110	Opens on Zone 1 pressure lower than Elev. 202 in Concrete Reservoir.[a]
V-PRV-5	Sparrow	River	2	1	30	8	132	Opens on Zone 1 pressure lower than Elev. 202 in Concrete Reservoir.[a]
V-PRV-6	32nd	Lake	2	1	40	6	109	Opens on Zone 1 pressure lower than Elev. 202 in Concrete Reservoir.[a]

[a] Concrete tank top hydraulic grade line is at elevation 211.

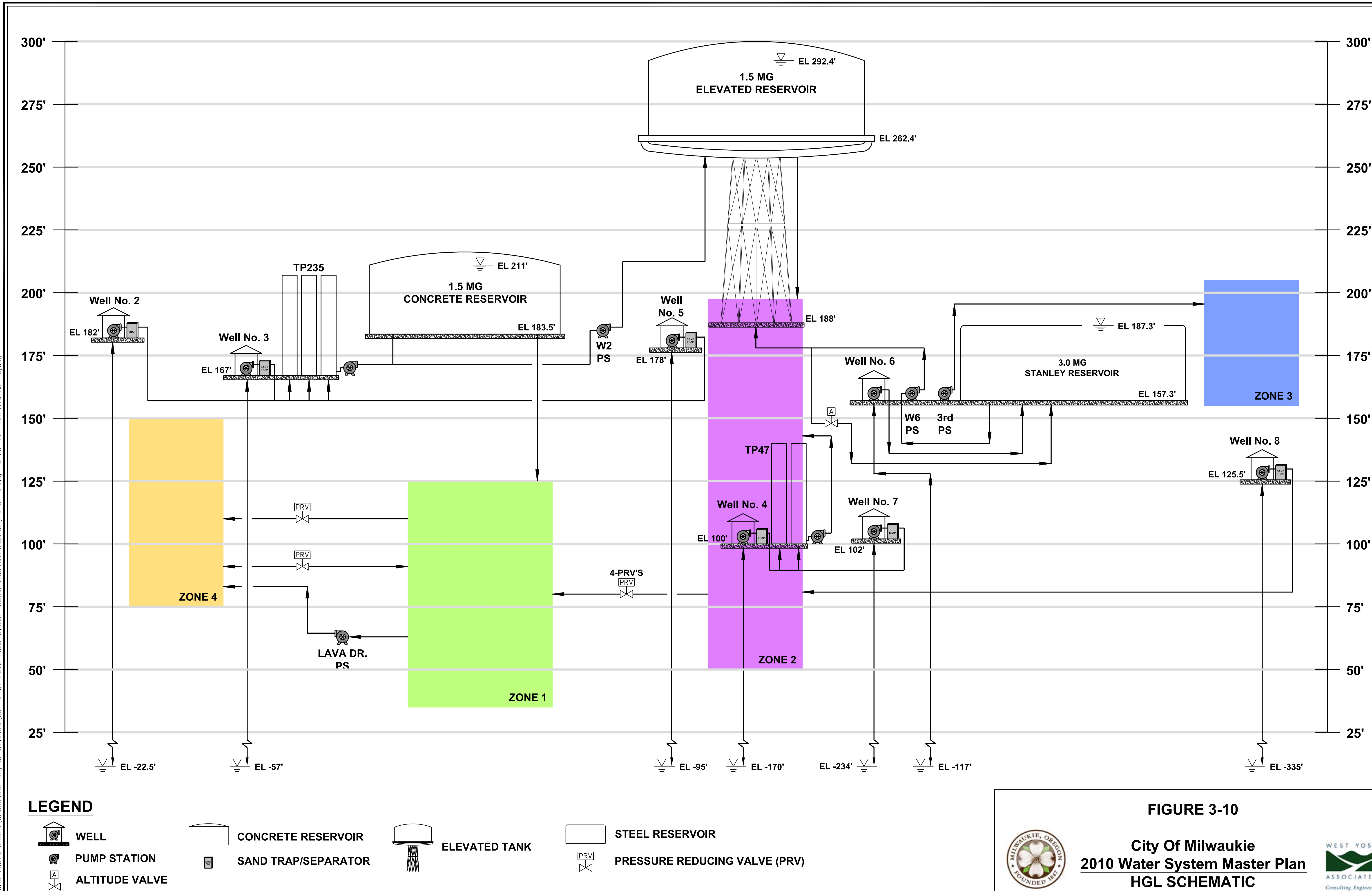

FIGURE 3-10

City Of Milwaukie
2010 Water System Master Plan
HGL SCHEMATIC

Water Demand

The purpose of this chapter is to present the current and projected potable water demands for the area served by the City. Projected water demands will be based on land use and unit demand factors for each type of land use. Reliable water demand estimates are necessary to:

- Develop and calibrate the water system hydraulic model
- Help identify deficiencies in the existing water system
- Assist in the assessment of future water system capacity
- Help identify and secure sufficient water supplies to serve customers under various hydrologic conditions
- Help develop the final capital improvement plan (CIP)

The following sections of this chapter describe the data and methodology utilized to determine the City's potable water system demands:

- Historical Water Production and Consumption
- Adopted Peaking Factors
- Projected Water Demands

4.1 HISTORICAL WATER PRODUCTION AND CONSUMPTION

Water production is the combined quantity of water produced by the City's groundwater wells and water received from adjacent water districts via water system interties. Water consumption is the quantity of water actually consumed or used by its customers. The difference between production and consumption is unaccounted-for water (UAFW).

The City currently measures all of the water produced by its wells, received from adjacent water purveyors, and used by all of its customers. Consequently, the City tracks water use in two ways: production records and meter (consumption) records.

4.1.1 Historical Water Production

The City meets its customers water demands with groundwater pumped from its own wells. Figure 4-1 presents the historical water production from 2000 to 2009. The production data was collected by the City and summarizes water production for the period including peak day and peak month data. Population data is based on Portland State University estimates. As shown in Figure 4-1, groundwater production varies year to year, but has not increased in the last 10 years.

Figure 4-2 compares total historical water production and historical average annual rainfall during the dry season. Water demand typically increases during the dry season for landscape irrigation. During the ten years of records shown, the highest production occurred in the years 2000 and 2003, which were also the driest years during this period. From 2005 through 2007, the water meter on Well No. 5 was not properly installed, and under-recorded the amount of water delivered by this well. During the two wettest summers in 2001 and 2004, the total annual production was lower than average. As shown in Figure 4-2, water production can vary as much as 50 MG between a dry summer and a wet summer.

4.1.2 Historical Water Consumption

Historical water consumed between 2005 and 2009, within each of the City's revenue classes, is summarized in Table 4-1. A review of the data from 2005 to 2009 indicates that every revenue class saw little variation over the past 5 years. The 2009 annual average water used was 2.4 mgd.

4.1.3 Historical Unaccounted-for Water

UAFW in the City is the difference between the recorded production from groundwater wells, including water from the CRW intertie and metered consumption.

UAFW is typically caused by uses such as hydrant testing, fires fighting, system flushing, system leaks, and water main breaks. Construction water use is typically captured in the "Other" revenue class. City water production data for 2005 to 2007 is understated since the water meter on Well No. 5 was not properly installed. For planning purposes in this Water System Master Plan, UAFW for the City is 11 percent based on 2008 and 2009 data.

4.1.4 Historical Per Capita Water Demand

Historical per capita water demands were calculated by dividing the total water production by the estimated historical population. Table 4-2 summarizes historical per capita water demands for the City between 2000 and 2009. As shown in Table 4-2, the historical average per capita water demand has remained relatively stable, averaging about 116 gallons per capita per day (gpcd) over the past 10 years. Water production for 2005 through 2007 was not used because the total was understated.

Figure 4-3 compares the historical per capita demand and historical population. As shown, population has increased at a relatively slow and constant rate from 2000 to 2009, and the per capita demand has hovered at around 116 gpcd. The majority of the variation observed in the per capita demand is tied to total water demand which appears to vary based on hydrologic conditions as discussed previously (*i.e.* quantity of rainfall).

Table 4-1. Historical Water Consumption by Revenue Class

Year	Metered Use, ccf			Total, ccf	Total Gallons			Total, Percent
	Residential	Multi unit Dwelling	Commercial		Metered use	Production (including CRW input)	Unaccounted for Water	
2005	613,487	104,335	257,883	975,705	729,827,340	801,168,000	71,340,660	8.9%
2006	651,740	140,758	258,805	1,051,303	786,374,644	832,098,000	45,723,356	5.5%
2007	612,446	193,712	267,061	1,073,219	802,767,812	821,170,000	18,402,188	2.2%
2008	604,105	145,811	286,976	1,036,892	775,595,216	869,321,000	93,725,784	10.8%
2009	594,472	153,750	298,095	1,046,317	782,645,116	879,066,000	96,420,884	11.0%
Average UAFW								11%

Source: From City Data: Item 8 Historical Max Day.xls and Item 9 Consumption Report 2005-2009.pdf

Data in shaded area understates production since the meter on Well 5 was not operating correctly.
ccf = 100 cubic feet

Table 4-2. Historical Per Capita Demand

Year	PSU Estimates[a]	Production, gallons	Per Capita Demand, gpcd
2000	20540	894,113,000	119
2001	20550	853,567,000	114
2002	20550	861,440,000	115
2003	20580	910,463,000	121
2004	20590	866,465,000	115
2005	20655	801,168,000	106
2006	20835	832,098,000	109
2007	20920	821,170,000	108
2008	20915	869,321,000	114
2009	20920	879,066,000	115
		Average[b]	116

[a] Source: From Portland State University Center for Population Research estimates and Item 8 Historical Max Day.xls
[b] Does not include per capita demand estimates from 2005 through 2007.
Data in the shaded area understates actual production since the meter on Well 5 was not operating correctly.

4.2 ADOPTED PEAKING FACTORS

Peaking factors are used to calculate water demands expected under high demand conditions (i.e., maximum day and peak hour demand). The resulting demand conditions for maximum day and peak hour periods are then used to evaluate and size transmission/distribution pipelines and storage facilities, and to define water supply needs and capacity requirements. This section describes the methodology used to develop the peaking factors for the maximum day and peak hour demand conditions within the City.

Table 4-3 summarizes the historical average day and corresponding maximum day peaking factors, between 1999 and 2009. As shown in Table 4-4, the maximum day peaking factor for the City has ranged from a high of 2.0 in 2000 to a low of 1.8 in 2007-2009. For planning purposes in this Water System Master Plan, a maximum day peaking factor of 1.9 was adopted. A peaking factor of 1.9 represents the average over the historical period from 1999 to 2009 and is consistent with the peaking factor observed in other communities.

Table 4-3 also summarizes four years of data that was collected on the year's peak day and analyzed for peak hour. While this data is limited to four years, it is representative of system peaks in recent years. The maximum peak hour occurred in 2007 and the minimum in 2008 corresponding to 2.9 and 2.6 respectively. The average of all three peaking factors is 2.7 and will be used for planning purposes in this Water System Master Plan.

Table 4-4 summarizes the maximum day and peak hour peaking factors adopted for this Water System Master Plan.

Table 4-3. Historical Maximum Day Peaking Factors					
Year	Average Day, mgd[(a)]	Maximum Day, mgd[(a)]	Peak Hour, mgd	Average Day to Maximum Day Peaking Factor	Average Day to Peak Hour Peaking Factor
1999	2.4	4.5		1.9	
2000	2.5	5.0		2.0	
2001	2.3	4.5		2.0	
2002	2.4	4.4		1.8	
2003	2.5	4.8		1.9	
2004	2.4	4.5		1.9	
2005	2.2	4.1		1.9	
2006	2.3	4.4	6.4	1.9	2.8
2007	2.3	4.1	6.6	1.8	2.9
2008	2.4	4.3	6.2	1.8	2.6
2009	2.4	4.4	6.3	1.8	2.6
			Average	1.9	2.7

[(a)] From City production data – includes unaccounted for water

Table 4-4. Adopted Peaking Factors	
Type of Factor	Adopted Factor
Average Day (ADD) to Maximum Day Demand (MDD)	1.9
Average Day to Peak Hour Demand (PHD)	2.7

4.3 PROJECTED WATER DEMANDS

Water demands were projected through buildout of the City using a unit demand methodology based on land uses in the Comprehensive Plan. A land use based methodology was used instead of a per capita demand methodology, because per capita water demand projections uniformly distribute water use over the entire water service area, and therefore, do not account for specific land uses and associated water demands in specific locations.

Subsequent sections describe the land use based methodology used, followed by a discussion of total projected water demands.

4.3.1 Unit Demand Factors Adopted for this Water System Master Plan

Unit demand factors from 2009 were determined using meter data, parcel data, and land use data obtained from the City. The water meter records were linked to parcels using addresses; 83 percent of all available water meter records were linked to parcels. Because the parcel data did not have a land use designation assigned to it, the Comprehensive Plan land use data was then used to assign a land use designation to each parcel. Figure 4-4 illustrates the methodology used to link Comprehensive Plan land use data to water meter records using parcel data. Using this procedure, the total calculated water use for the year was within one percent of the actual water used that is shown in Table 4-1. Given this result, the unit demand factors provide a representative tool for estimating water demands from undeveloped areas.

The unit demand factor for each land use designation was calculated by dividing the total water use by the total parcel area for which it was linked; however, the parcel area used in this initial calculation did not include streets (see blue area on Figure 4-3) and therefore, represented net area. Accordingly, the unit demand factors calculated were net unit demand factors.

The net unit demand factors were used to project future demands by multiplying the appropriate net unit demand factor by the future acreage. However, acreage for future developments is gross area and therefore, includes the streets. Typically, the net unit demand factor would not be used to calculate demands for gross areas. In order to be consistent with the use of the same unit demand factor for existing and future developments and to provide additional conservatism for planning level purposes in this water system, the net unit demands factors were used to project future demands.

A normalization factor of 1.03 was used to adjust the net unit demand factors to account for variation in customer water use from year to year. Since the City net unit demand factors were developed based on one year of data (2009), the normalization factor was applied to each unit demand factors. In the last five years, the annual water used in 2007 was three percent higher than 2009; the normalization factor accounts for this difference.

Figure 4-4. Illustration of Unit Demand Factor Methodology

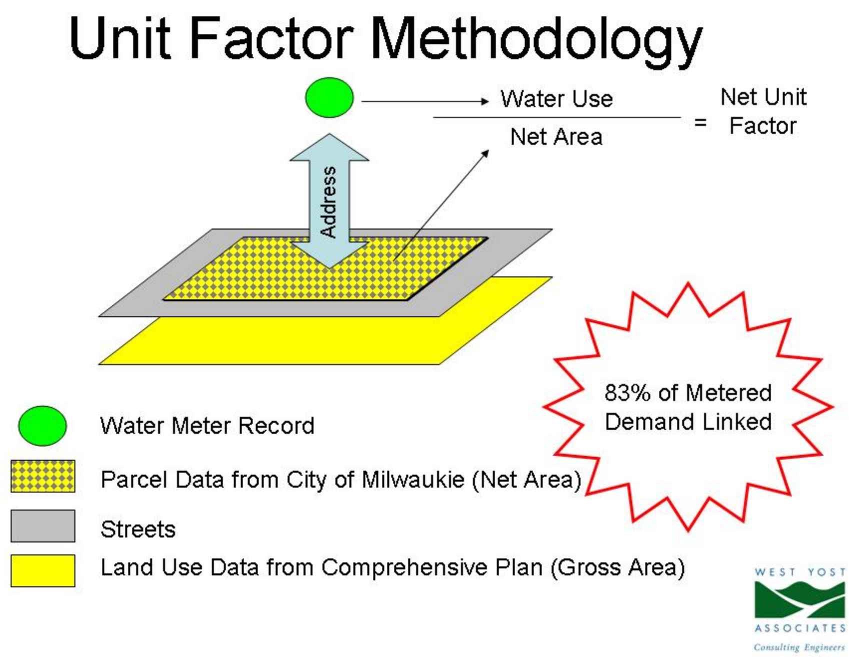

Table 4-5 summarizes the acreages of each existing land use designation within the City limits. The water unit demand factor for each land use category is summarized in Table 4-6. Table 4-6 also includes recommended unit demand factors for future planning. These planning level demand factors allow for more intensive water consumption patterns in the future.

Table 4-5. Land Use in Acres

Land Use Category	2009 Served Area[a], acres	Percentage
Low Density Residential	1,029	48
Moderate Density Residential	219	10
Medium Density Residential	83	4
High Density Residential	182	8
Commercial	57	3
Mixed Use (Commercial/High Density Residential)	29	1
Industrial	373	17
Public	143	7
Town Center	45	2
Total	2,159	100%

(a) Area based on City lots data (citylots09.shp) within the City Limits that are linked to 2009 billing data.

Land Use Category	2009 Water Use[(a),] gpd	2009 Served Area, acres	Calculated Unit Demand Factor, gpd/acre	Normalized Unit Demand Factor[(b)], gpd/acre
Low Density Residential	1,091,625	1,029	1,061	1,093
Moderate Density Residential	245,623	219	1,122	1,156
Medium Density Residential	145,350	83	1,760	1,813
High Density Residential	152,653	182	840	865
Commercial	72,810	57	1,279	1,317
Mixed Use (Commercial/High Density Residential)	26,834	29	919	947
Industrial	345,254	373	924	952
Public	29,248	143	205	211
Town Center	59,401	45	1,332	1,372
Total	2,168,798[(c)]	2,159		

Table 4-6. Summary of Recommended Unit Water Demand Factors

(a) Does not include unaccounted for water.
(b) Equal to the calculated unit demand factor multiplied by the normalization factor of 1.03. This factor was calculated using the maximum total metered use over the past five years, which was equal to 2.20 mgd in 2007 divided by the total metered used from 2009 (2.14 mgd).
(c) Annual demand within one percent of actual water used as shown in Table 4-1.
gpd = gallons per day

4.3.2 Future Development & Annexation

Future increases in water demand in the City will occur in two ways, infill development and annexation. The available area for future infill development was determined by using the current City vacant land inventory and categorizing the developable land by land use designation.

The City's UGMA lies within the Metro Regional Urban Growth Boundary (UGB) and is shown in Figure 4-5. This is the area outside of the current City limits that is planned for future annexation into the City. Most of the UGMA is unincorporated Clackamas County. A small section of the City of Portland and Happy Valley also lie within the City's UGMA. Expansions to the City's UGMA are not anticipated in the foreseeable future. The actual timing of annexation for lands within the UGMA is uncertain and will likely proceed on an ad hoc basis. For the purposes of this report, water demands for buildout of the full UGMA are being evaluated as they relate to the City's ability to supply the area and to help guide policy decisions regarding annexation.

Dual Interest Areas A and B are smaller subsections of the City's UGMA located within Clackamas County. These areas are adjacent to current City limits and have been identified as areas likely to be annexed into the City. The process to annex properties in Dual Interest Area A has already begun as part of the Northeast Sewer Extension project. As a result of the present and future annexations, the City must be prepared to provide future water service to these identified Dual Interest Areas.

Annexation of the UGMA and Dual Interest Areas includes adding both the existing developed areas and future infill development within these areas. The existing developed areas of the UGMA and Dual Interest Areas was determined and categorized by land use designation. Future infill development in the UGMA and Dual Interest Areas was determined by using the current vacant land available for development and categorizing by land use designation.

4.3.3 Projected Water Demands

Total projected water demands at buildout for the City were calculated by multiplying the recommended unit demand factors (see Table 4-6) by the additional developed acreage projected to occur as shown in Table 4-7. The resulting projection was added to existing 2009 water demands followed by adjustments for UAFW of 11 percent. Table 4-8 summarizes the total projected water demand for the City.

At buildout, the City's total average day demand for the existing service area increases to 2.5 mgd. If both Dual Interest Areas are added to the City water system, the total average day demand will increase to 2.8 mgd. The City average water demand could increase by 204 percent (7.3 mgd) with the annexation of the entire UGMA.

Table 4-9 summarizes the current and buildout water demands for the City's current service area, the dual interest areas and the urban growth management areas. The buildout demand for the existing service area will only increase by about four percent since most of the area is developed. Serving both Dual Interest Area A and B will add an average demand of 300,000 gpd or 13 percent.

Water demand for the UGMA would more than double the existing water demand in the City. The existing average demand in the City is 2.4 mgd while the demand for the UGMA has been estimated at 4.2 mgd. This average demand is based on land use and has not been confirmed through an analysis of the billing records for CRW.

Table 4-7 Projected Water Demands

Land Use Designation	Normalized Unit Demand Factor, gpd/ac	Additional Acreage			Additional Consumption, gpd
		Vacant	Existing	Total	
Within City Limit					
Low Density Residential	1,093	41.0		41.0	44,813
Moderate Density Residential	1,156	6.9		6.9	7,976
Medium Density Residential	1,813	13.9		13.9	25,201
High Density Residential	865	6.2		6.2	5,363
Commercial	1,317	0.1		0.1	132
Mixed Use (Commercial/ High Density Residential)	947	2.0		2.0	1,883
Industrial	952	11.9		11.9	11,329
Public	211	0		0	0
Town Center	1,372	8.4		8.4	11,388
Subtotal		90.4		90.4	108,084
Dual Interest Area A[a]					
Low Density Residential[b]	1,093	7.4	117.3	124.7	136,297
Medium Density Residential[c]	1,813		2.3	2.3	4,170
Commercial	1,317		1.9	1.9	2,502
Industrial	952	2.0	10.8	12.8	12186
Subtotal		9.4	132.3	141.7	155,155
Dual Interest Area B[a]					
Low Density Residential[b]	1,093	17.7	79.3	97.0	106,021
Subtotal		1,717	79.3	97.0	106,021
Urban Growth Management Area[a]					
Low Density Residential[b,d]	1,093	148	1,718	1,866	2,039,538
Medium Density Residential[c]	1,813	11	235	246	445,998
Commercial	1,317	69	675	744	979,848
Industrial	952	54	318	372	354,144
Forest	0	0	4	4	0
Mixed Use Community (MUC)	1,317	0	187	187	246,279
Planned Open Space	0	107	23	130	0
No Water Use (PGE Property, Easement, Park and Ride, Rail Road)	0	5	151	156	0
Subtotal		394	3,311	3,705	4,065,807
Total		2,210.8	3,522.6	4,034.1	4,435,067

(a) Land use file for Dual Interest Areas A and B, and the UGMA are obtained from the City.
(b) Includes Single Family (SFR) and Rural Residential (RUR) land use types.
(c) Includes Multi Family Residential (MFR) land use type.
(d) Includes Agriculture Area (AGR) land use type.

Table 4-8. Summary of Future Average Water Demand by Area

Description	Within City Limits	Dual Interest Areas		UGMA	Total
		A	B		
Additional Consumption at Buildout	108,084	155,155	106,021	4,065,807	4,435,067
Unaccounted for Water	11,889	17,067	11,662	447,239	487,857
Total Additional Demand	119,973	172,222	117,683	4,513,046	4,922,924
Existing Demand	2,408,400	--	--	--	2,408,400
Total Future Demand	2,528,373	172,222	117,683	4,513,046	7,331,324
Increase	5%	--	--	--	204%

Table 4-9. Water Demand Projections, mgd

Demand	Current Service Area		Dual Interest Area A		Dual Interest Area B		UGMA		Total	
	2009	Buildout	2009	Buildout	2009	Buildout	2009	Buildout	2009	Buildout
Average Day	2.4	2.5	0.2	0.2	0.1	0.1	4.2	4.5	6.8	7.3
Maximum Day	4.6	4.8	0.3	0.3	0.2	0.2	7.9	8.6	13.0	13.9
Peak Hour	6.5	6.8	0.4	0.5	0.3	0.3	11.2	12.2	18.4	19.8

Figure 4-1. Comparison of Annual Water Production and Population

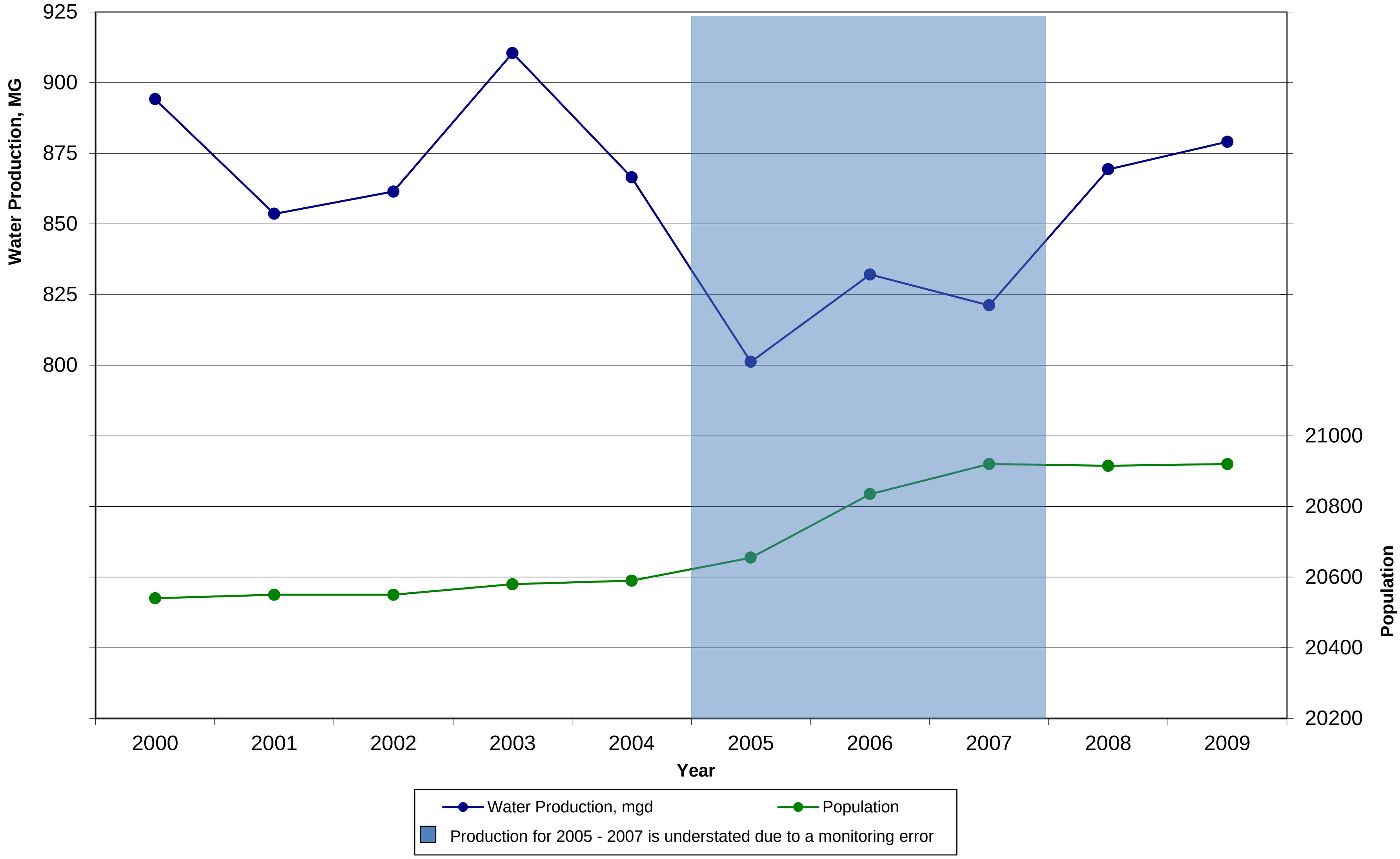

Figure 4-2. Comparison of Water Production and Average Dry Season Annual Rainfall

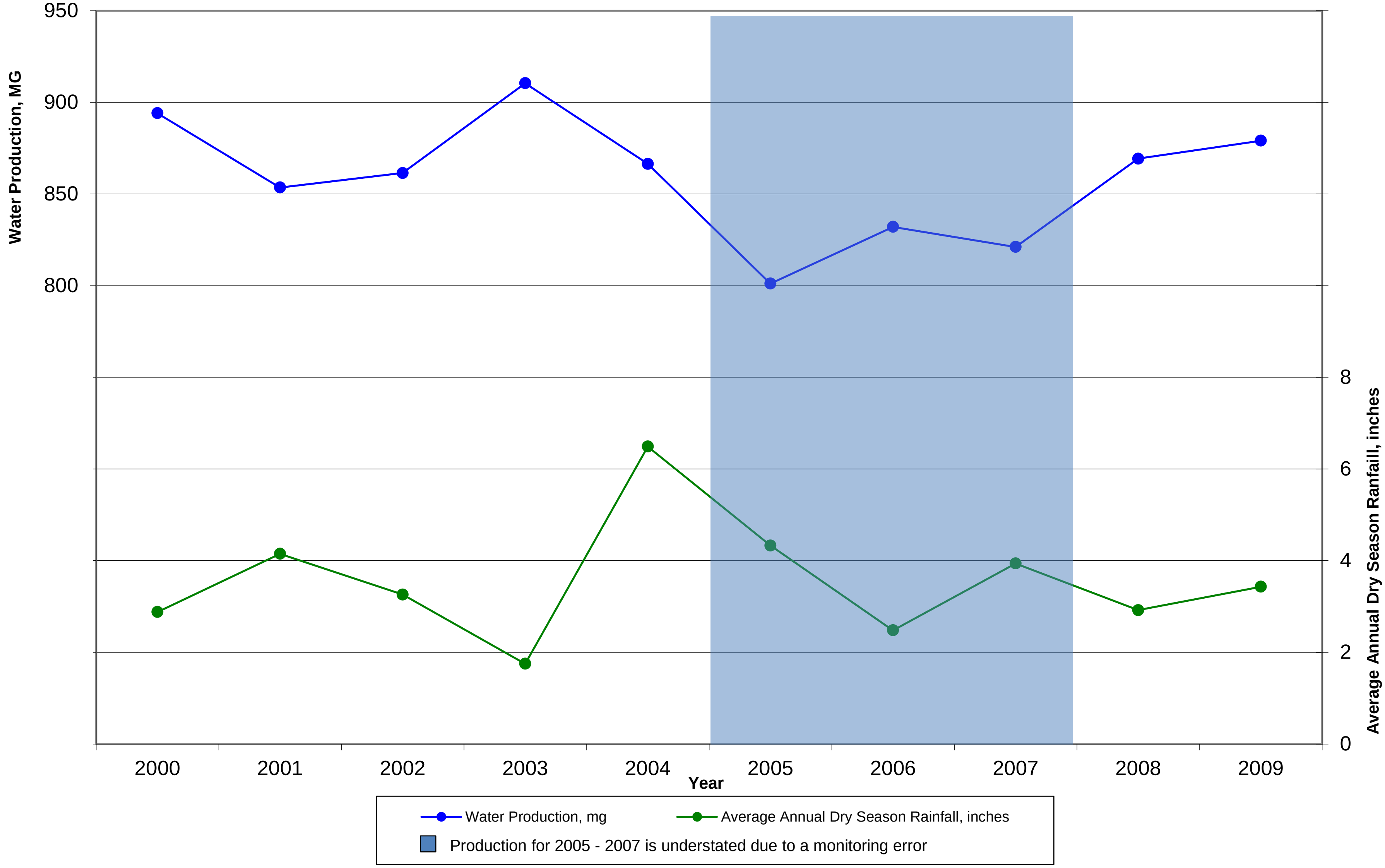

Figure 4-3. Comparison of Historical Per Capita Demand and Population

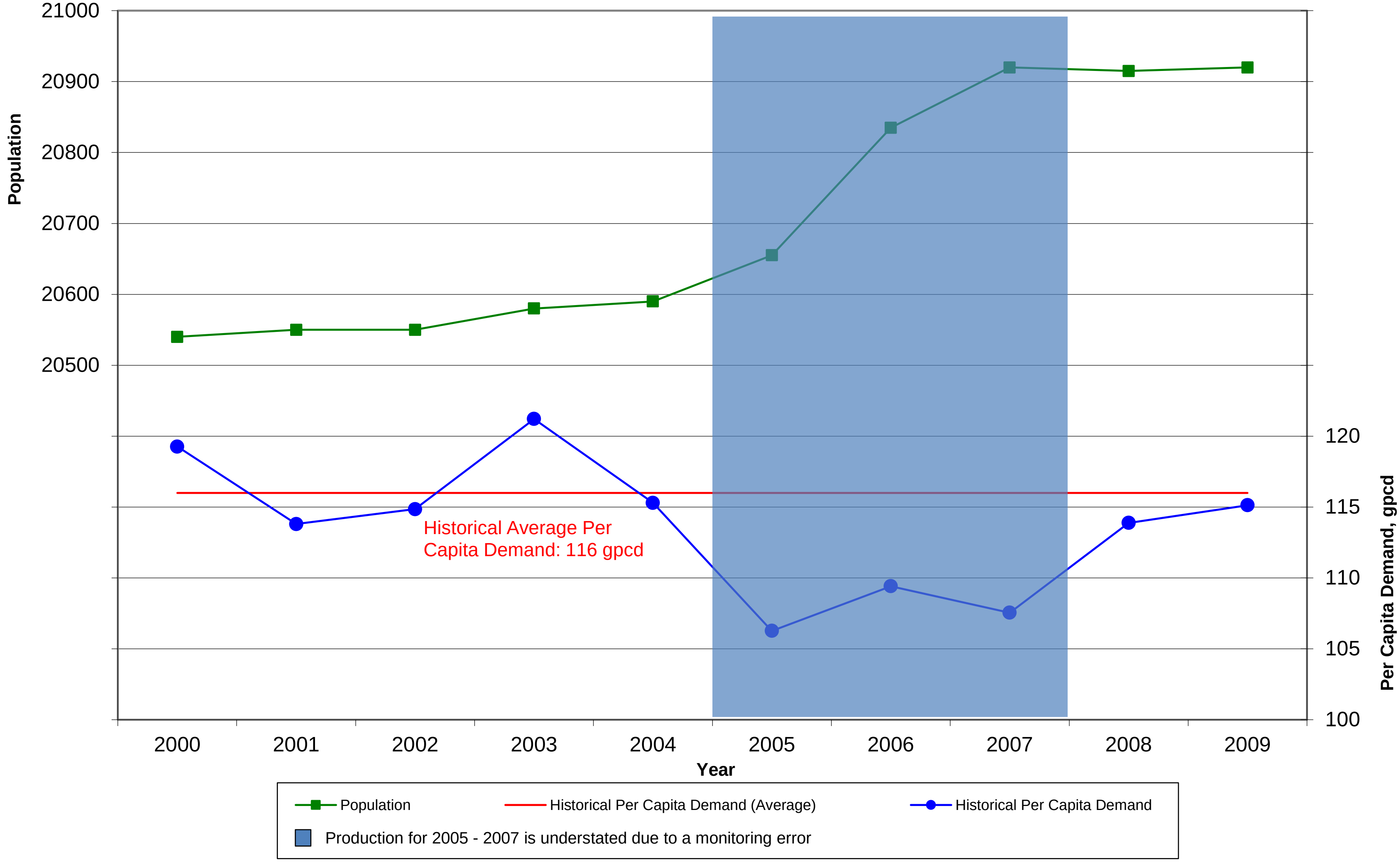

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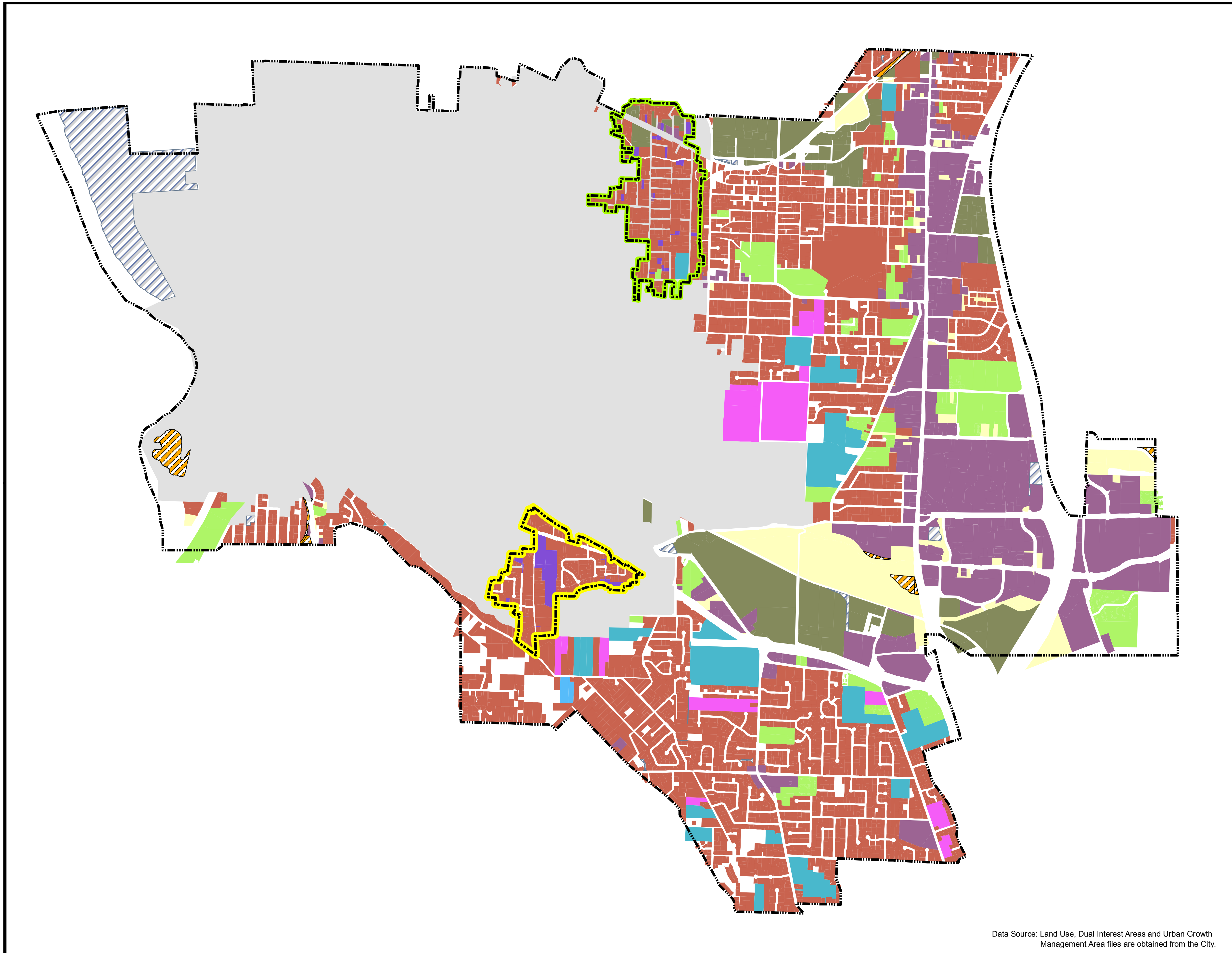
FIGURE 4-5

City of Milwaukie
2010 Water System Master Plan

FUTURE GROWTH



0 1,100 2,200
Scale in Feet

LEGEND

- City of Milwaukie
- Dual Interest Area A
- Dual Interest Area B
- Urban Growth Management Area

Land Use Category

- Low Density Residential
- High Density Residential
- Commercial
- Agriculture
- Forest
- Industrial
- Rural
- Mixed Use
- Park / Open Space
- Water From City of Portland and Wells

WEST YOST ASSOCIATES
Consulting Engineers

CHAPTER 5
Water Distribution System Service Standards

The purpose of this chapter is to define the water distribution service standards for analyzing the performance of the City's potable water distribution system. The service standards recommended in this chapter provide a basis for evaluating the City's existing water distribution system and guide the planning and design of those improvements to the water system that are necessary to meet future demands. These standards include the desired fire flow and flow duration, definition of "emergency events", pumping capacity, storage capacity components (including operational, fire flow and emergency), minimum and maximum system pressures, and maximum pipeline velocity and head loss. The water distribution system service standards used for this WMP are summarized in the following sections:

- General Water System Reliability
- Fire Flow Requirements
- Water System Conditions During High Demand
- Pumping Facility Capacity
- Critical Pumping Facilities
- Water Storage Capacity
- Water Transmission and Distribution System
- Water System Standards Summary

5.1 GENERAL WATER SYSTEM RELIABILITY

Attention to enhancing the reliability of the system under all conditions is an important part of maintaining high quality water service. Water system reliability is achieved through a number of system features including (1) appropriately sized storage facilities, (2) redundant or "firm" pumping, transmission, and treatment facilities where required, and (3) alternate power supplies. Reliability and water quality are also improved by designing looped water distribution pipelines and avoiding dead-end distribution mains whenever possible. Looping pipeline configurations reduces the potential for stagnant water and the associated problems of poor taste and low chlorine residuals. In addition, proper valve placement is also necessary to maintain reliable and flexible system operation under normal and abnormal operating conditions.

5.2 FIRE FLOW REQUIREMENTS

While the City is the purveyor of water, the Clackamas County Fire District #1 (CCFD) is also concerned with the availability of adequate water supply. The City is responsible for supply and distribution of water; whereas, CCFD establishes minimum water flows required for firefighting purposes.

CCFD uses the 2010 Oregon Fire Code Table B105.1 *Minimum Required Fire-Flow and Flow Duration for Buildings* to assist them in establishing minimum fire flows and durations for individual structures.

The City's minimum design standards for fire flow are 1,000 gpm for a one or two-family dwellings (which is consistent with the minimum requirements of the CCFD), 3,000 gpm for a commercial building, and 5,000 gpm for buildings in heavy commercial areas. However, actual fire flow requirements are ultimately determined by CCFD and Insurance Service Office (ISO) on a case-by-case basis. Specific fire flow requirements are based on the size of building (in square feet) and type of construction (wood frame, metal, masonry, installation of sprinklers, *etc*.). Once the fire flow requirement is established, it is multiplied by the required duration to determine the total volume needed for fire flow storage.

Table 5-1 represents the general fire flow requirements that have been established for planning the City's water system. Construction type and fire flow area are not generally known during the development of a master plan; consequently, fire flow requirements set forth in Table 5-1 are based on previous estimates for these land use types in similar communities to the City. In all land use types, they are at or above the minimum criteria set forth on the 2010 Oregon Fire Code.

5.3 WATER SYSTEM CONDITIONS DURING HIGH DEMAND

In accordance with typical industry standards, the City's water supply system should have the capability to meet a system demand condition equal to the occurrence of a maximum day demand condition concurrent with a fire flow event. For planning purposes, it is assumed that the maximum day plus fire flow demand condition will consist of a single concurrent fire flow event.

5.3.1 Water Supply

The reliable yield of all sources of water supply shall exceed the projected maximum day demand on the system. The definition of reliable yield of water supplies is the total potable water production and delivery capacity of the water system during the worst drought. The worst drought conditions are estimated from historical stream flow records. Generally, it is recommended that the total maximum production capacity be at least ten percent greater than the maximum day demand to allow for concurrent fire flow demands, assure compliance with drinking water quality standards during periods of poor source water quality, and repair of water system equipment.

5.3.2 System Pressure Requirements

Under normal operating conditions, water pressure in the distribution system should range between 40 and 100 psi. The lower end of this pressure range is intended to ensure that adequate pressure is available for the highest fixture at a service connection during maximum demand conditions. The higher end of this pressure range is intended to minimize system repairs, lower the potential for surge damage, minimize water leakage rates, and reduce pressure rating of pipes, thus reducing the cost of new pipeline installation.

Under fire flow conditions, lower pressures in the distribution system are allowable. In accordance with Oregon State Department of Human Services (DHS) rules, the minimum system pressure under fire flow conditions shall be 20 psi as measured at the property line.

Table 5-1. Recommended Fire Flow Requirements[a,b]

Designation	Non-Sprinklered			Sprinklered[c,d]		
	Fire Flow, gpm	Duration, hours	Recommended Storage, MG	Fire Flow, gpm	Duration, hours	Recommended Storage, MG[e]
Single-Family Residential[f]	1,500	2	0.18	--	--	--
Multi-Family Residential[g]	1,500	3	0.27	--	--	--
Institutional[h]	3,000	4	0.72	2,000[i]	4	0.36
Industrial/Commercial[j]	5,000	4	1.20	3,000[i]	4	0.60

(a) Construction type and fire area are not generally known during the development of a master plan; consequently, fire flow requirements set forth in this table are based on previous estimates for these land use types and similar communities.

(b) Unique projects or projects with alternate materials may require higher fire flows and will be reviewed by the Fire Marshal on a case-by-case basis (e.g., proposed commercial/industrial areas and schools).

(c) The Fire Marshal normally allows up to a 50 percent reduction in fire flows if a building is sprinklered. However, the Fire Code also requires that no fire flow be less than 1,000 gpm for single family residential or 1,500 gpm for all other building types. For a more conservative fire flow estimate, Single Family and Multiple Family buildings were considered non-sprinklered for this Water Master Plan Update.

(d) Specific fire flows were determined from Table B105.1 of the 2007 OFC, and depend on construction type and fire area. These fire flow requirements are based on buildings being fully sprinklered.

(e) Recommended storage volumes do not include volume associated with 500 gpm sprinkler flow.

(f) Single Family includes Low Density Residential and Medium Density Residential land use.

(g) Multiple Family includes High Density Residential land uses.

(h) Institutional includes Parks & Recreation and Public and Quasi-Public land uses.

(i) Fire flow includes a 500 gpm demand for on-site sprinkler flow.

(j) Industrial/Commercial includes Commercial, Mixed Use Corridor, Mixed Use Downtown, Mixed Use Employment, Industrial and Future Urban land uses.

5.4 PUMPING FACILITY CAPACITY

Sufficient water system pumping capacity should be provided to meet the greater of these two demand conditions:

1. A maximum day demand concurrent with the largest single fire flow requirement in the pressure zone with the largest pump at each booster pump station in standby mode.
2. A peak hour demand with the largest pump at each booster pump station in standby mode.

5.5 CRITICAL PUMPING FACILITIES

Critical pumping facilities are defined as those pumping facilities that provide water to service area(s) without sufficient emergency storage (see emergency storage section) and that meet the following criteria:

- The largest pumping facility that provides water;
- A pumping facility that provides the sole source of water to a single or multiple pressure zone(s); and
- A pumping facility that provides water from a supply well.

All critical pumping facilities should be equipped with an on-site, back-up power generator. At less critical facilities, a plug-in adapter will be used to allow interconnection to a portable generator, which will be brought to the site by City staff during a prolonged power outage.

If unavailable by gravity storage, the fire flow should be supplied with a National Fire Protection Association (NFPA) rated fire pump. If an NFPA rated fire pump is not used, then a pump(s) and motor(s) combination with a back-up power source of sufficient capacity to meet the required maximum fire flow and minimum residual pressure requirements, as determined by the CCFD's Fire Marshal, is required. Pump stations serving pressure zones with elevated storage for pressure control are controlled using pressure control valves.

5.6 WATER STORAGE CAPACITY

Standards have been developed for determining treated water storage capacity needs within the individual pressure zones of a distribution system to meet diurnal operational peaks and emergency conditions. Per AWWA Manual 32, storage requirements can generally be categorized into the following four components:

- Operational Storage
- Equalization Storage
- Fire Flow Storage
- Emergency Storage

The following discussion presents design guidelines for each of these four components.

5.6.1 Operational Storage

The operational storage component allows for the continued supply of water to the system from reservoirs during temporary shutdowns of the water treatment plants or booster pump stations. The necessary volume of operational storage is determined based on the anticipated timing and duration of temporary shutdowns during the maximum demand period. As a result, the necessary operational storage volume is dependent on the layout and functions of each water system utility and is widely variable from system to system. Because the City's treatment plants and booster pumping stations are capable of operating as long as necessary during the maximum demand period, there is no need for dedicated operational storage within the City's distribution system.

5.6.2 Equalization Storage

Over any 24-hour period, water demand on the distribution system will vary. Typically, water demand will be high in the morning when people are getting ready for the day, then will decline to a nominal baseline level that is dominated by the water use patterns of commercial and industrial areas. Demand will then begin to increase again in late afternoon, reaching a higher level in the early evening as people return home from work. During periods when the rate of demand exceeds the wells' production rate, the excess demand is provided from equalization storage. During periods when the rate of demand is less than the treatment plant's production rate, the equalization storage is recharged. When a typical diurnal demand pattern is compared to the average daily demand, the necessary supply from equalization storage is typically equal to 25 percent of daily demand. Therefore, to ensure the availability of adequate equalization storage during a maximum day demand event, equalization storage requirements should be 25 percent of the maximum day demand.

5.6.3 Fire Storage

Generally, fire flows will be provided by storage. Fire flow storage for each pressure zone must be provided by the reservoir(s) that serve that pressure zone. The necessary fire flow storage for each pressure zone is determined by the highest fire flow requirement of that pressure zone multiplied by the required duration the flow is to be maintained. Pumped fire flows are allowed for small areas where the pump station provides an adequate firm capacity, sufficient pressure, and reliable operation. These areas would be small, isolated zones where construction of a gravity storage facility is not practical.

5.6.4 Emergency Storage

A reserve of treated water is also required to meet demands during emergency outage periods, when normal supply is interrupted. An emergency is defined as an unforeseen or unplanned event that may degrade the quality or quantity of potable water supplies available to serve customers. There are three types of emergency events that a water utility typically prepares for:

- Minor emergency. A fairly routine, normal, or localized event that affects few customers, such as a pipeline break, malfunctioning valve, hydrant break, or a brief power loss. Utilities plan for minor emergencies and typically have staff and materials available to correct them.
- Major emergency. A disaster that affects an entire, and/or large, portion of a water system, lowers the quality and/or quantity of the water, or places the health and safety of a community at risk. Examples include water treatment plant failures, raw water contamination, or major power grid outages. Water utilities infrequently experience major emergencies.
- Natural disaster. A disaster caused by natural forces or events that create water utility emergencies. Examples include earthquakes, forest or brush fires, hurricanes, tornados or high winds, floods, and other severe weather conditions such as freezing or drought.

Determination of the required volume of emergency storage is a policy decision based on the assessment of the risk of failures and the desired degree of system reliability. The amount of required emergency storage is a function of several factors including the diversity of the supply sources, redundancy and reliability of the production facilities, and the anticipated length of the emergency outage. In developing an emergency storage requirement for the City, typical industry standards were used.

The American Water Works Association (AWWA) states that no formula exists for determining the amount of emergency storage required, and that the decision will be made by the utility based on a judgment about the perceived vulnerability of the system. For this Water System Master Plan, it has been assumed that the emergency storage requirement will be based on minor emergencies and specific major emergency criteria. Based on this assumption, and the fact that the City does have emergency supply connections with adjacent agencies, it is recommended that the City have a minimum quantity of emergency storage volume equivalent to the average day demand.

5.6.5 Total Water Storage

The minimum treated water storage capacity in the system available to each pressure zone shall equal the sum of the following:

- Operational. The minimum operational storage is based on the layout and functions of the individual water system utility. Because the City’s treatment plants and booster pumping stations are capable of continuous operation during the maximum demand period, dedicated operational storage is not required.
- Equalization. The storage allocated for meeting diurnal demand peaks should be equivalent to 25 percent of the maximum day demand. This storage volume should be located within the pressure zone.

- Fire Flow. The storage allocated to provide fire flows should be equivalent to the maximum fire flow in the pressure zone multiplied by the duration the flow rate must be maintained.
- Emergency. The minimum emergency storage volume allocated for providing water during periods when normal supply is interrupted is based on the water system vulnerability, or the frequency and duration of water service interruption. Typically, the minimum emergency storage volume should be equivalent to 100 percent of the average day demand.

5.6.6 Reservoirs

Reservoir facilities are sized in accordance with the total water storage capacity required in each pressure zone. Reservoir inlet and outlet piping shall be designed to facilitate adequate turnover of stored water at the facility and avoid water quality problems. Reservoir management techniques such as lowering reservoir levels during periods of low demand will also ensure the freshness of the water supply and eliminate the need for rechlorination.

To ensure adequate service pressures, new reservoirs are typically placed so that the overflow elevation is 100 feet above the normal upper service elevation of the pressure zone it is serving. This arrangement will allow for fluctuations in reservoir level while maintaining system pressures within the desired range. The City should consider equipping reservoirs with a remote-controlled shut-off valve or seismic valve to prevent drainage after a significant earthquake.

5.7 WATER TRANSMISSION AND DISTRIBUTION SYSTEM

The following criteria will be used as guidelines for sizing new distribution pipelines. However, the City's existing system will be evaluated on a case–by-case basis. For example, if an existing pipeline experiences head loss in excess of the criteria described below during a maximum day plus fire flow event, this condition, by itself, does not necessarily indicate a problem as long as the minimum system pressure criterion is satisfied.

Consequently, the City's existing system will be evaluated using pressure as the primary criterion; and secondary criteria, such as pipeline velocity, head loss, age, and material type, will be used as indicators to locate where water system improvements may be needed.

New transmission and distribution pipelines to serve the City's future planning areas should be located within designated utility corridors wherever possible. These designated utility corridors should be within public rights-of-way to minimize or eliminate the need for utility easements within private property.

5.7.1 Pipeline Networks

The pipelines in the City's distribution system will generally be sized based on the criteria described below for average, maximum day and peak hour demand conditions.

5.7.1.1 Average Day Demand

- Pressures should be maintained between a maximum of 100 psi and a minimum of 40 psi.
- The maximum velocity within the distribution system pipelines should be 3 to 5 feet per second (fps).

5.7.1.2 Maximum Day Demand plus Fire Flow

- The minimum allowable residual pressure should be 20 psi at the flowing fire hydrant.
- The maximum velocity within the distribution system pipelines should be 10 fps.
- Head losses within the distribution system pipelines should be limited to 10 feet per thousand feet (ft/kft) of pipeline.

5.7.1.3 Peak Hour Demand

- The minimum allowable service pressure should be 40 psi.
- The maximum velocity within the distribution system pipelines should be 7 fps.
- Head losses within the distribution system pipelines should be limited to 10 ft/kft of pipeline.

The distribution system shall be looped at all possible locations to maintain adequate circulation and water quality. Long, dead-end pipelines shall be avoided whenever possible to prevent water quality problems. When unavoidable, a fire hydrant or blow-off hydrant shall be installed at the end of the line to facilitate periodic system flushing. A maximum development size of 25 lots will be allowed on a dead-end line.

5.7.2 Valves

Valve location and spacing are important considerations in the design of a water distribution system. Pipelines must include an adequate number of properly located valves to allow for isolation of pipeline sections in the event of maintenance operations or new construction. Typical industry standards for valve spacing are identified in Table 5-2. The supply pipelines that deliver water to the City's system are those coming out of the wells and to the treatment facilities. The transmission and distribution pipelines provide the network grid from which most customer connections are served. A general guideline for locating valves in the distribution system is that smaller branch mains should be equipped with a valve so that any service problems on the branch pipeline do not require a shut-off of the major transmission line. Within the distribution grid, placement of a valve on all legs of tees and crosses will minimize the extent of a service disruption during system work. For the same reason of localizing service disruptions, system design should avoid direct service taps into transmission pipelines whenever possible.

Table 5-2. Maximum Valve Spacing Standards	
Pipeline Function	Maximum Spacing
Supply pipeline	1 mile
Transmission pipeline	1,300 feet
Residential distribution pipeline	800 feet
Commercial distribution pipeline	500 feet

5.7.3 Hydrants

Fire hydrants are dispersed throughout the distribution system to provide the emergency flows required for fire protection. The requirements for spacing fire hydrants are defined in Appendix C – Fire Hydrant Location and Distribution of the Oregon Fire Code, and are shown in Table 5-3. In applying the fire code, the CCFD shall determine the required fire hydrant distribution based on their judgment.

Table 5-3. Number and Distribution of Fire Hydrants

Fire Flow Requirement, gpm	Minimum Number of Hydrants	Average Spacing Between Hydrants[a,b,c], feet	Maximum Distance from any Point on Street or Road Frontage to a Hydrant[d]
1,750 or less	1	500	250
2,000 – 2,250	2	450	225
2,500	3	450	225
3,000	3	400	225
3,500 – 4,000	4	350	210
4,500 – 5,000	5	300	180
5,500	6	300	180
6,000	6	250	150
6,500 – 7,000	7	250	150
7,500 or more	8 or more[e]	200	120

For SI: 1 foot = 304.8 mm, 1 gallon per minute = 3.785 L/m

(a) Reduce by 100 feet for dead-end streets or roads.

(b) Where streets are provided with median dividers which cannot be crossed by fire fighters pulling hose lines, or where arterial streets are provided with four or more traffic lanes and have a traffic count of more than 30,000 vehicles per day, hydrant spacing shall average 500 feet on each side of the street and be arranged on an alternating basis up to a fire-flow requirement of 7,000 gallons per minute and 400 feet for higher fire-flow requirements.

(c) Where new water mains are extended along streets where hydrants are not needed for protection of structures or similar fire problems, fire hydrants shall be provided at spacing not to exceed 1,000 feet to provide for transportation hazards.

(d) Reduce by 50 feet for dead-end streets or roads.

(e) One hydrant for each 1,000 gallons per minute or fraction thereof.

In general, no hydrant shall be installed on a water main with less than an 8-inch inside diameter and the hydrant shall have a minimum 6-inch inside diameter. However, in certain cases where it is proven that the hydrant and distribution main can meet flow and pressure requirements, connection to a water main with a 6-inch inside diameter will be allowed. Hydrants shall be located as close to the distribution main as possible and shall be no more than 40 feet away. To comply with this requirement, hydrants will generally be located on the same side of the street as the distribution main. In areas where required fire flows exceed 1,500 gpm, the water supply must be provided by more than one hydrant (see Table 5-3).

5.8 WATER SYSTEM STANDARDS SUMMARY

A summary of the recommended potable water system performance and operational criteria is presented in Table 5-4 and reflect typical water system industry standards, including the DHS, Oregon Administrative Rules (OAR), the Environmental Protection Agency (EPA), the AWWA, the ISO, and the Oregon Fire Code (OFC).

Table 5-4. City of Milwaukie Planning and Design Criteria

Component	Criteria	Remarks / Issues
PERFORMANCE CRITERIA FOR PLANNING & DESIGN		
Fire Flow Requirements (flow [gpm] @ duration [hours])		
Single-Family Residential	1,500 gpm @ 2 hrs	Fire flows based on new development requirements. Existing development will be evaluated on a case by case basis, because of the historical varying standard.
Multi-Family Residential	1,500 gpm @ 3 hrs	
Institutional (schools, hospitals, etc.)	2,000 gpm @ 4 hrs (with approved automatic sprinkler system)	
Commercial/Industrial	3,000 gpm @ 4 hrs (with approved automatic sprinkler system)	
Water Supply Capacity		
Maximum Day Demand Plus Fire Flow	Provide capacity equal to maximum day demand plus fire flow	
Peak Hour Demand	Provide capacity equal to peak hour demand	
Pumping Facility Capacity		
Booster Pump Capacity	Equal to the maximum day demand for the pressure zone.	Design for maximum day plus fire flow or peak hour (whichever is larger), only if no gravity storage is available within the pressure zone and/or service area.
Backup Power	Equal to the firm capacity of the pumping facility.	On-site generator for critical stations.[a] Plug in portable generator for less critical stations.
Water Storage and System Peaking Capacity		
Equalization	25 percent of maximum day demand	
Fire	Varies (see requirements listed in remarks column)	Varies depending on required fire flow duration. Highest fire flow demand in any particular area controls size of required storage (see Table 4-2). Recommended fire storage volume does not include volume associated with 500 gpm sprinkler flow. 1,500 gpm @ 2 hrs = 0.18 MG 1,500 gpm @ 3 hrs = 0.27 MG 2,500 gpm @ 4 hrs = 0.60 MG
Emergency	Maximum day demand	Based on DHS recommendations.
Total Water Storage Capacity	Equalization + Fire + Emergency	
Water Transmission Line Sizing		
Diameter	18-inches in diameter or larger	
Average Day Demand Condition		Criteria based on requirements for new development, existing transmission mains will be evaluated on case-by-case basis. Evaluation will include age, material type, velocity, head loss, and pressure.
Minimum Pressure [psi]	40 psi	
Maximum Pressure [psi]	100 psi	
Maximum Velocity [ft/sec]	3 fps	
Maximum Day Demand Condition		
Minimum Pressure [psi]	40 psi	
Maximum Head loss [ft/1000 ft]	3 ft/kft	
Maximum Velocity [ft/sec]	5 fps	
Peak Hour Demand Condition		
Minimum Pressure [psi]	40 psi	
Maximum Head loss [ft/1000 ft]	3 ft/kft	
Maximum Velocity [ft/sec]	5 fps	
Hazen Williams "C" Factor	140	For consistency in hydraulic modeling.
Pipeline Material	Ductile Iron	
Water Distribution Line Sizing		
Diameter	Less than 18-inches in diameter	Must verify pipeline size with max day and fire flow analysis.
Average Day Demand Condition		Criteria based on requirements for new development, existing distribution mains will be evaluated on case-by-case basis. Evaluation will include age, material type, velocity, head loss, and pressure.
Minimum Pressure [psi]	40 psi	
Maximum Pressure [psi]	100 psi	
Maximum Velocity [ft/sec]	3 - 5 fps	
Maximum Day w/ Fire Flow Demand Condition		
Minimum Pressure [psi] (at fire node)	20 psi	
Maximum Head loss [ft/1000 ft]	10 ft/kft	
Maximum Velocity [ft/sec]	10 fps	
Peak Hour Demand Condition		
Minimum Pressure [psi]	40 psi	
Maximum Head loss [ft/1000 ft]	10 ft/kft	
Maximum Velocity [ft/sec]	7 fps	
Hazen Williams "C" Factor	140	For consistency in hydraulic modeling.
Pipeline Material	Ductile Iron	
Maximum Valve Spacing		
Supply Pipeline	1 mile	
Transmission Pipeline	1,300 feet (minimum)	
Residential Distribution Pipeline	800 feet	
Commercial Distribution Pipeline	500 feet	
Uniform Fire Code Hydrant Distribution Requirements		
Residential	500	
Commercial, Industrial, and Other High Value District	200-500	
OTHER CRITERIA		
Maximum Number of residential lots that can be served by a non-looped water pipeline	25 lots	If a non-looped water line goes out-of-service, all associated residences lose water service.

[a] A pumping facility is defined as critical if it provides service to pressure zones and/or service areas without sufficient emergency storage and that meet the following criterion:

- The largest facility that provides water to a particular pressure zone and/or service area;
- A facility that provides the sole source of water to single or multiple pressure zones and/or service areas; and
- A facility that provides water from a supply turnout into pressure zones and/or service areas.

CHAPTER 6
Hydraulic Model Development

This chapter describes the development, calibration, and verification of the City's water distribution system hydraulic model.

To develop the City's hydraulic network model, West Yost completed the following steps:

- Used City's existing water distribution system maps (exported from City's GIS) to create the hydraulic model,
- Verified that the hydraulic model system configuration (pipeline sizes, alignments, connections, and other facility sizes and locations) is representative of the current City's water system,
- Allocated existing water demands by using City's spatially located account information to distribute demands within the hydraulic model, and
- Calibrated the City's water system hydraulic model to simulate pressures and flows observed in the field.

To accomplish these tasks, West Yost worked closely with City's Engineering and Operations staff to obtain and review:

- Available information regarding existing transmission and distribution mains, storage tanks, groundwater wells, pump stations and other water facilities,
- As-built drawings and maps detailing sections of the system to confirm pipeline sizes, material type, age, locations and alignments, and
- Available metered account data.

The water distribution system model was then calibrated and verified using tank level, flow, and pressure data observed in the field during July 2010. The hydraulic model development, calibration, and verification are described in the following sections.

6.1 DEVELOPMENT OF THE HYDRAULIC MODEL

West Yost developed a hydraulic model of City's water system using a series of steps that included the following:

- Incorporated the description of the model and element definitions
- Imported pipelines, nodes, and junctions into InfoWater
- Assigned roughness factors in InfoWater
- Allocated elevations in H_2OMAP
- Spatially located accounts in GIS
- Allocated water demands in H_2OMAP
- Incorporated station elements into InfoWater
- Applied naming scheme in InfoWater

Each of these steps is discussed in more detail below.

6.1.1 Description of the Model and Element Definitions

MWH Soft's InfoWater is the hydraulic modeling software used to represent the City's water system. This computer simulation model transforms information about the physical system into a mathematical model that solves for various flow conditions based on specified demands. The computer model then generates information on pressure, flow, velocity and head loss that is used to analyze system performance and to identify system deficiencies. The model can also be used to verify the adequacy of recommended or proposed system improvements.

The hydraulic model is represented as a skeletonized network of nodes (*e.g.*, location of a tank, location where pressure is monitored), and node-connecting elements (*e.g.*, pipes). However, because nodes are representative of various actual facilities (*e.g.*, tanks, pump stations, or wells) and physical locations, a definition of each element was created during the development of the hydraulic model. The description of nodes and node-connecting elements are described as follows:

<u>Node</u>: Nodes, as defined for the City's model, represent transitions in pipeline characteristics (*e.g.*, diameter) or points in the system where pressure or water quality is monitored. Nodes also represent locations in the system where metered water demands do not exist, such as at wells, pump station and tanks. Elevation and physical facility location are the data requirements for nodes.

<u>Junction</u>: Junctions, as defined for the City's model, represent locations in the system where water is subtracted from the system and are used in the model to mark the locations in the system where a water demand exists. Junctions can also include transitions in pipeline characteristics (*e.g.*, diameter). Data requirements for junctions are the demand at each junction, elevation and location.

<u>Pipe</u>: Pipes (*i.e.*, links), as defined for the City's model, represent facilities that convey water from one point in the system to another and are used to represent pipelines or check valves in the model. Diameter, from/to node or junction, length and pipeline roughness factor are the input data required.

<u>Reservoir</u>: Reservoirs represent external sources of water for the model (*e.g.*, groundwater basin), and remain at a constant level irrespective of the flow unless they are specified as variable-head reservoirs. Reservoirs are used to represent the source for each of the groundwater wells in the City's model. Location, water surface elevation, and nominal pressure are the input data required.

<u>Tank</u>: Tanks, as defined for the City's model, are distinguished from reservoirs by having known finite volumes and water surface elevations that change with time as water flows into or out of the facility. This element is used to represent the City's storage tank. Diameter, bottom elevation, overflow elevation, and location are the input data required.

Pump: Pumps, as defined for the City's model, represent locations in the model where the hydraulic grade line is raised to overcome elevation differences and friction losses, and are used to represent pump stations. Elevation, number of pumps, pump test results, pump curves, sequencing, and location are the input data required.

Valve: Valves, as defined for the City's model, regulates either flow or pressure in the distribution system. Diameter, setting, elevation and location are the input data required.

6.1.2 Pipelines, Nodes, and Junctions Imported into InfoWater

City staff provided a GIS geodatabase file containing the geospatial location of existing pipelines, check valves, and control valves for the City water system. The geodatabase layer of the existing water pipelines was imported into the hydraulic model, but did not include "from" and "to" nodes (*i.e.*, points designating the beginning and end of the pipeline). Consequently, InfoWater's *Append Nodes* feature was used to create and assign the beginning and end-points (from and to nodes) for the existing pipelines. In addition, West Yost also developed an attribute in the hydraulic model database to include the original GIS geodatabase Object ID, allowing City staff to leverage or integrate information with City's GIS.

6.1.3 Roughness Factors Assigned in InfoWater

The original geodatabase layer for existing water pipelines did not include roughness factors. However, the geodatabase did include material type, which is an attribute that can be used as a surrogate for roughness factor. Consequently, West Yost assigned a preliminary roughness factor (*i.e.*, C-factor), based on experience and professional judgment, to each pipeline by using its material type and year of construction as a surrogate in the hydraulic model. Table 6-1 presents the C-factors assigned to each of the different material types within the City's water system. These C-factors were then validated during calibration of the hydraulic model.

6.1.4 Elevations Allocated in H_2OMAP

Light Detection and Ranging (LIDAR) topography was received from the City, and was used to assign elevations to each node using H_2OMAP's Elevation Interpolation feature. Certain service elevations (at the existing pump stations) were later confirmed during calibration.

6.1.5 Accounts Spatially Located in GIS

This section describes the methodology used to spatially locate water consumption for the metered accounts which include Residential, Multi-Density Residential and Commercial.

Table 6-1. C-Factors Assigned in the Model[a, b]														
	Cast Iron (CI)		Ductile Iron (DI)		Galvanized Steel (GALV)		Polyvinyl Chloride (PVC or C900)		Steel (STL)		Steel Lined (STLRNF)		Unknown	
Year	Diameter ≤ 8-inches	Diameter > 8-inches	Diameter ≤ 8-inches	Diameter > 8-inches	Diameter ≤ 8-inches	Diameter > 8-inches	Diameter ≤ 8-inches	Diameter > 8-inches	Diameter ≤ 8-inches	Diameter > 8-inches	Diameter ≤ 8-inches	Diameter > 8-inches	Diameter ≤ 8-inches	Diameter > 8-inches
1930-1940	80	110	100	120	NA	NA	NA	NA	NA	NA	NA	NA	110	120
1941-1950	90	110	100	120	NA	NA	NA	NA	NA	85	NA	NA	110	120
1951-1960	100	120	100	120	100	NA	140	150	NA	85	NA	NA	110	120
1961-1970	110	120	110	120	100	NA	140	150	NA	85	NA	NA	120	130
1971-1980	110	120	120	130	110	NA	140	150	NA	100	NA	NA	120	130
1981-1990	120	130	120	130	120	NA	140	150	NA	130	NA	NA	130	130
1991-2000	120	130	130	140	130	NA	140	150	NA	140	NA	NA	130	130
2001-2010	120	130	130	140	140	NA	140	150	NA	140	NA	140	130	130

[a] Acronym obtained from geodatabase layer, PIPETYPE, for City of Milwaukie
[b] NA - Not Applicable, material was not installed during these years

6.1.5.1 Metered Accounts

The City provided billing spreadsheets containing metered accounts and their corresponding metered consumption data by address and customer class for each month of the year from 2005 through 2009. The most recent data set from 2009 was used to develop the existing water demands for the hydraulic model and is the baseline for projecting future demands in Chapter 4.

Consumption data from metered accounts was spatially located using two separate methods. The primary method is linking the consumption data by address to a separate GIS parcel file. Once no additional matches are found, the secondary method is applied, which is geocoding any remaining consumption data using a GIS street file. This secondary method assigns the billing data to the centerline of the street for which its address corresponds. Figure 6-1 illustrates the methodology used to link the addresses associated with the consumption data to the addresses in the GIS parcel file and street file.

West Yost was able to spatially locate 99.5 percent of the metered accounts (6,750 out of 6,783) present in the 2009 billing spreadsheet provided by the City. Although there is no minimum industry standard for geocoding, this amounts to 98 percent of the total 2009 metered consumption within the City. The remaining consumption in the City was either from meters without proper addresses or meters with addresses that did not match up with either the GIS parcel file or the GIS street file. Table 6-2 presents the percentage of total metered accounts and metered consumption spatially located for the City.

Table 6-2. Spatially Located Results for City of Milwaukie

Category	Number of Metered Accounts	Total "Metered" Demand, afa	Average Day Demand, gpm
Actual 2009[a]	6,783	2,922	1,810
Spatially Located using Parcel file[a]	6,578	2,793	1,730
Spatially Located using Street file	172	82	58
Spatially Located Total[b]	6,750	2,875	1,788
Percent of Actual 2009	99.5%	98%	98%

(a) Data provided by City in May 2010 and does not include unaccounted-for water.
(b) Based on West Yost's GIS.

Figure 6-1. Illustration of Methodology for Spatially Locating Metered Accounts

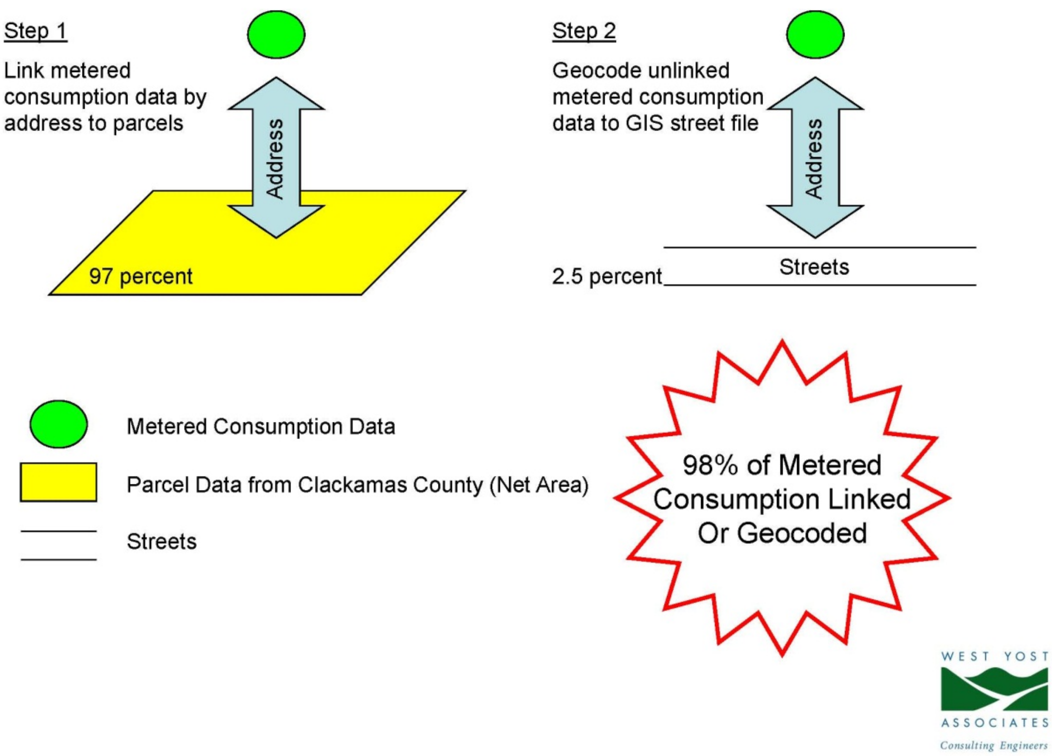


Figure 6-2 compares the spatially located water demand data with existing pipelines imported into InfoWater. As shown in Figure 6-2, most areas with spatially located demands also had an existing pipeline. This correlation indicates that the geodatabase layer used as the basis for the hydraulic model includes most of the existing pipelines.

6.1.6 Water Demands Allocated in H_2OMAP

For the City's water system, water demands were allocated in the hydraulic model using the spatially located demand data developed in the previous section and the Demand Allocation/Pro module of H_2OMAP (Allocation Module). The Allocation Module has six fully automated methods for accurately computing and loading network models based on demand type, location and variation. The method used for the City's model was the "Closest (nearest) Pipe Method." This method locates the closest pipeline to each meter (*i.e.* if there are parallel pipelines at the meter point, the demand will be allocated to the pipeline that is closest to its position). Demands are then assigned to the closest or furthest junction node on either side of the pipeline based on a distance-weighted approach. West Yost staff reviewed the model after running the Allocation Module to confirm that the demands were allocated to the correct pipeline.

Water demand within the hydraulic model was allocated to each revenue class designation, providing City with additional flexibility in the model. Table 6-3 presents the demand column assigned to each revenue class within the hydraulic model.

Table 6-3. Revenue Class Assignment

Customer Class Description[a]	Demand Column in Model[b]
Residential Metered	1
Multiple Residential Metered	2
Commercial Metered	3

(a) Customer class description provided by City in May 2010.
(b) Column number corresponds to Demand # Column in Junction database in the InfoWater model.

6.1.7 Station Elements Incorporated into InfoWater

After the nodes and pipelines were imported into the hydraulic model, major system facilities (*e.g.*, groundwater wells, pump stations, and storage tanks) were digitized into the model. Each of these facilities was entered into the model by hand based on drawings provided by the City.

6.1.8 Naming Scheme Applied in InfoWater

After the major facilities were digitized into the model, each model element was assigned a label which identifies the type of model element, the element's purpose, and the element's location. Assigning each model element a specific label allows the modeler to easily locate specific elements or more readily identify potential problems during the calibration and verification process. The City model was populated using the naming scheme presented in Table 6-4.

6.2 DIURNAL CURVE DEVELOPMENT

To add the time variable to the City's hydraulic model and to create a true extended period simulation (EPS) model, West Yost developed a representative 48-hour diurnal pattern for the City's service area.

The extended simulation is based on station SCADA data on tank level, flows, and pump discharge pressures for the City's tank, wells and booster pump stations. This information was obtained for the period from July 6, 2010 to July 11, 2010. July 10 and 11, 2010 were selected as the period when recorded flow characteristics best represented system operations. Consequently, hourly production data from the tank, wells and booster pump station were summed using SCADA flow recordings to represent the total demand for each City's Pressure Zone. By using a 48-hour demand pattern, the model can more accurately represent fluctuations in demand over the simulation period.

Table 6-4. Naming Scheme for Network Elements

Model Component	Naming Scheme
Pipelines	P-1-100 “100” = Sequential Number “1” = Pressure Zone 1 “P” = Pipeline
Junctions	J-1-100 “100” = Sequential Number “1” = Pressure Zone 1 “J” = Junction
Nodes	N-1-100 “100” = Sequential Number “1” = Pressure Zone 1 `“N” = Node
Wells	W-02 “02” = Station Number “W” = Reservoir
Well Pumps	PMP-W-1 “1” = Well Number “W” = Well Station “PMP” = Pump
Tanks	T-CONCRETE “CONCRETE” = Station Name “T” = Tank
Booster Pumps	PMP-LAVA-HDP1 “HDP” = High Demand Pump “1” = Sequential Number “LAVA” = Facility Station Name “PMP” = Pump
Valves	V-LAVA-1 “01” = Sequential Number “LAVA” = Facility Station Name “V” = Valve V-PRV-1 “01” = Sequential Number “PRV” = Pressure Reducing Valve “V” = Valve

There are 4 pressure zones in the City service area. All pressure zones are supplied water from groundwater wells. The following paragraphs provide the methodology used in the development of the diurnal curve for each of the City's pressure zones.

6.2.1 Pressure Zone 1

The main source of supply in Pressure Zone 1 is the Concrete Tank which is supplied by Wells 2, 3 and 5. Flow information from Wells 2, 3 and 5 is available from SCADA. The Concrete Tank level information is also available from SCADA, and was converted to flow data by using the volume of the tank. At Well 2 facility site, a transfer pump station (W2 Transfer Pump Station) is used to move water from the Concrete Tank to the Elevated Tank. Flow from SCADA for the W2 Transfer Pump Station is also available.

There are three PRVs that provide supplemental supply from Pressure Zone 2 to Pressure Zone 1. These PRVs are regulated based on pressure. There is no recorded flow available for these PRVs. Based on the recorded field pressure and SCADA information for the Concrete Tank, and Wells 2, 3 and 5, there were flow through these PRVs when pressure in the Pressure Zone 1 system dropped below the pressure setting at these PRV stations. However, flows are minimal, and the amounts of these flows were not able to be verified since there were no SCADA available at these PRV stations. Consequently, flow through each PRV for development of the diurnal curve was assumed to be zero.

Pressure Zone 1 also provides supply to Pressure Zone 4 through the Lava Pump Station. The flow at the Lava Pump Station is recorded on SCADA.

To create the Pressure Zone 1 diurnal curve, the flow from the elevated tank was either added or subtracted, depending on if the tank was emptying or filling, respectively, from the total production from Wells 2, 3 and 5 at an hourly increment over a period of 48 consecutive hours from July 10 to 11, 2010. In addition, the flows through the Lava Pump Station and the W2 Transfer Pump Station were subtracted from the total production from Wells 2, 3 and 5. The resulting normalized diurnal pattern is provided in Figure 6-3.

6.2.2 Pressure Zone 2

Pressure Zone 2 is supplied from the Elevated Tank. There are 3 main groundwater wells that provide supply into the Elevated Tank. These wells are Wells 4, 7 and 8. Flow information for these wells is available from SCADA. The Elevated Tank level information is also available from SCADA, and was converted to flow data by using the volume of the tank.

Pressure Zone 2 also provides supply to Pressure Zone 1. Water can be conveyed from Pressure Zone 2 to Pressure Zone 1 through 3 PRVs as previously described in Section 6.2.1. In addition, the W2 Pump Station conveys water from the Concrete Tank to the Elevated Tank. Flow from SCADA for this station is available.

An altitude valve facility located at Well 6 site is used to move water from the Elevated Tank to the Stanley Tank. This altitude valve may provide additional supply from Pressure Zone 2 to Pressure Zone 3. A valve at Well 6 must be manually operated to allow supply from Pressure Zone 2 to Pressure Zone 3. Once the valve is operated, the water systems can be monitored by

SCADA. For the time period selected for development of the diurnal curve, SCADA information for this facility indicates zero flow.

Pressure Zone 2 can also receive water from Well 6 through the W6 Transfer Pump Station. The SCADA information for the W6 Transfer Pump Station indicates zero flow during the two-day time frame.

The diurnal curve for Pressure Zone 2 was calculated by either adding or subtracting, depending on whether the Elevated Tank was emptying or filling, respectively, from the total production from Wells 4, 7 and 8. The flow from the W2 Pump Station was added to the Pressure Zone 2 diurnal curve. The resulting normalized diurnal pattern is shown in Figure 6-4.

6.2.3 Pressure Zone 3

Pressure Zone 3 is supplied from the Stanley Tank through the Zone 3 Pump Station. Well 6 is the main groundwater well that supplies the Stanley Tank. Flow information from this Zone 3 Pump Station is available from SCADA and was compiled at one hour increments over a period of 48 consecutive hours to develop a diurnal pattern. Figure 6-5 illustrates the normalized diurnal pattern for Pressure Zone 3. For the time period selected for development of the diurnal curve, SCADA showed that there was no water transfer to or from Pressure Zone 2.

6.2.4 Pressure Zone 4

Pressure Zone 4 is supplied from the Lava Pump Station. Flow information from this pump station is available from SCADA, and was compiled at one hour increments over a period of 48 consecutive hours to develop a diurnal pattern. Figure 6-6 illustrates the normalized diurnal pattern for Pressure Zone 4. Approximately 20 gpm flows continuously from Pressure Zone 4 to Pressure Zone 1 to maintain fresh water in Pressure Zone 4.

6.3 HYDRAULIC MODEL CALIBRATION

The City’s hydraulic model was calibrated to confirm that the computer simulation model can accurately represent the operation of the water distribution system under varying conditions. Calibration of the hydraulic model used data gathered through hydrant tests, optional fire flow tests, and hydrant pressure recorders, as described in the following sections.

6.3.1 Development of Hydrant (C-Factor) Tests

After developing the hydraulic model, locations were chosen for possible hydrant flow testing (see Figure 6-7). The selection of these hydrant test sites was based on specific pipeline size, material type and age. These hydrant tests were used to evaluate pipeline friction factors (C-factors) and to calibrate the model to ensure that the hydraulic model closely represented actual observed pressure conditions in the field.

Hydrant flow testing was scheduled and performed on July 8 and 9, 2010. Table 6-5 provides the field status of each hydrant test. Of the original 17 scheduled hydrant tests, 13 were performed in the field. Four hydrant tests were canceled due to constraint(s) identified by City staff. Each hydrant test involved flowing water through pipelines of a specific size, material type and age, and then measuring the pressure drops along the pipelines to determine friction losses. The hydrant test procedure consisted of monitoring discharge flow and pressure at the key flowing hydrant, and pressures at other hydrants along the supply routes to that key hydrant. Static pressures were measured while the key hydrant was closed, and residual pressures were measured while the key hydrant was flowing.

Table 6-5. City of Milwaukie Hydrant Test Locations and Status[(a)]

Test #	Diameter, inches	Material	Year Installation	Address	Field Status
1	8	CIP	1952	Along Cambridge Lane, South of Wavery Drive	Cancelled, Golf Tournament
2	6	CIP	1960	Along Clatsop Street, West of McLoughlin Boulevard	Cancelled Due to Portland's Main Break on McLoughlin
3	10	DIP	1980	Along SE Mailwell Drive, East of SPT Corr.	Completed
4	6	CIP	1930	Along Madison Street, West of 30th Avenue	Completed
5	8	DIP	1981	Along Milwaukie Marketplace	Completed
6	12	DIP	1979	Along SE International Way	Completed
7	12	CIP	1965	Along Mallard Way	Completed
8	10	CIP	1968	Near Linwood Elementary School, North of Grove Loop	Completed
9	6	DIP	1985	Along 66th Avenue, North of Eunice Street	Completed
10	8	DIP	1970	Along Linwood Avenue, North of Furnberg Street	Completed
11	6	CIP	1958	Along Fieldcrest Drive	Completed
12	8	CIP	1969	Along Filbert St, East of 32nd Avenue	Completed
13	6	PVC	1993	Along Sherrett Street	Completed
A1	12	CIP	1969	Between 17th Avenue and McBrod Avenue, South of Ochoco Street	Cancelled Due to Portland's Main Break on McLoughlin
A2	8	CIP	1952	Along McBrod Avenue, South of Ochoco Street	Cancelled Due to Portland's Main Break on McLoughlin
A3	6	DIP	1990	Along Pennywood Drive, West of Freeman Road	Completed
A4	8	DIP	1970	Along Johnson Creek Boulevard, Southeast of 45th Place	Completed

(a) 13 Test Locations (#1-13), 4 Alternate Test Locations (#A1-A4)

Pipelines in the City's water system range in size from 2-inches to 18-inches in diameter. Pipeline materials consist mainly of cast iron. Other pipeline materials as listed in Table 6-1 are also found in the City's water system. The pipeline age in the City's water system is 80 years old or newer.

Prior to the model runs, each pipeline was assigned a preliminary C-factor based on the pipeline size and material type as presented in Table 6-1. Consequently, each hydrant flow test was then simulated using the hydraulic model of the City's water system. Results were compared to the field data to determine the accuracy of the model. The differences between observed static and residual pressures for the field hydrant test, compared to readings predicted by the model, were calculated. Although no specific criteria for calibration of hydraulic models exist in the United States, the AWWA Engineering Computer Applications Committee (ECAC) has developed suggested guidelines. Based on these suggested guidelines, the goal of the calibration effort was to achieve no greater than a 5 psi differential between the field hydrant test data and model-simulated results. Results from the hydrant tests are discussed in more detail in the following section.

6.3.2 Hydrant (C-factor) Test Results

The results of the simulated hydrant flow tests generally validated the system pipeline configuration and confirmed the preliminary C-factors presented in Table 6-1. However, based on the comparison of the collected hydrant flow test data and model simulation results, one of the hydrant flow test (Test 9) required further evaluation because it did not meet the ±5 psi tolerance limit established for calibration.

The results from the remaining hydrant tests indicate that the hydraulic model accurately simulated the City's water system, and was able to closely replicate field-observed pressures and flows. The detailed result of each individual hydrant test that was performed is provided in Appendix A. Further discussion regarding Test 9 is provided below.

6.3.2.1 Hydrant Test 9: 6-inch DI Pipeline Constructed in 1985

The difference between measured and modeled pressures for Hydrant 9A in Test 9 was 14 psi. However, the C-factor for 6-inch DI pipelines is reasonable for this pipeline diameter, material and age. Therefore, the results from the hydraulic model simulation indicate that for Test 9 there are system configuration issues (*e.g.*, partially closed valve(s), inaccurate representation of pipeline connectivity or pipeline diameter).

There are two potential partially closed valves in the vicinity of Test No. 9 that warrant additional field investigation by the City to confirm the status of these valves. Location 1 is along Montgomery Drive, east of Linwood and location 2 is along Linwood, north of Furnberg Street.

For Test 9, it is recommended that City staff first confirm the valve status at the two locations. When either of these two values were assumed to be partially closed in the hydraulic model, Test 9 simulated within a 5 psi differential from the field hydrant test data.

6.3.3 Development of the Verification Process

Verifying that a hydraulic model replicates field conditions requires representation of how the system performs over a wide range of operating conditions. To ensure that the hydraulic model is correctly configured and capable of producing results that are consistent with those observed in the field, a verification process was conducted. Hydrant pressure recorders (HPRs) were used to record pressures in the field. The data were then compared with model-predicted pressures at the same system locations. Other pressure points monitored by City were also used in the verification process. A brief description of the verification process is presented below.

Sixteen hydrant pressure recorders were placed at different locations within the City's water distribution system. Each HPR collected field-pressure data for approximately five days (from July 6, 2010 to July 11, 2010). The locations were selected based on their proximity to the transmission mains and to extreme elevations (low and high) in the water distribution system. Since the City has 4 pressure zones, each pressure zone was assigned with at least 1 HPR. Figure 6-8 shows the location of each HPR. HPR 11 was missing data due to mechanical failure on the recorder. However, the absence of data from HPR 11 does not compromise the verification process because there are 9 other HPRs in Pressure Zone 2 which recorded field pressure during this time period.

Following the integration of the diurnal pattern into the hydraulic model, an EPS modeling run was performed and the resulting pressures at each of the HPRs, the flows and pressures at each well, and the tank level were graphed. To verify whether the City's hydraulic model was accurately predicting field-observed tank level, flows, and pressures, model-predicted tank level, flows, and pressures were compared to actual field data. Results from the verification process are discussed below.

6.3.4 Verification Results

Graphs of representative comparisons between model-simulated and field-observed tank level, flows, and pressures are provided in Figures 6-9 through 6-15. The following sections describe the verification results for each City's Pressure Zone.

6.3.4.1 Pressure Zone 1

Verification results for the City's water system indicate that the model-simulated tank level for Zone 1 trends well as shown in Figure 6-9. The model simulated flow at Wells 2, 3 and 5 facilities are slightly higher than the recorded SCADA flow. This slight discrepancy might be due to the water that is added from the Treatment Plant for Wells 2, 3 and 5 as part of the chlorine solution that is not recorded on the SCADA system. The overall model results for Pressure Zone 1 indicate that the model was able to replicate field conditions.

As illustrated in Figure 6-10, the simulated pressures for the pressure recorders in the City's Pressure Zone 1 trend closely to the recorded field pressure readings. Individual graphs of comparisons between model-simulated and field-observed pressures for each HPR are provided in Appendix B. These results indicate that the water demands are properly allocated in the model and that the modeled pipeline network is accurately configured.

6.3.4.2 Pressure Zone 2

Figure 6-11 illustrates verification results for the City's Pressure Zone 2. Results indicate that the model-simulated tank level for Zone 2 trends closely to the SCADA recorded level. Well 4 model-simulated flow trend fairly close to the SCADA recorded flow. The SCADA for Well 7 shows hourly fluctuation in flow, however Well 7 is controlled by the level in elevated storage reservoir and so these fluctuations do not seem consistent with how this Well operates. The model simulates that once the well is on it operates at a set flow and pressure. The model is consistent with the high points of the SCADA data, which indicates that we are simulating the operation of the facility correctly. We recommend that the City review the SCADA data for Well 7 to evaluate where the flow is being recorded. Well 8 model-simulated flow also trends close to the SCADA recorded flow. The overall model results for Pressure Zone 2 indicate that the model was able to closely replicate field conditions. Detailed verification results for other Pressure Zone 2 facilities are provided in Appendix B.

As illustrated in Figure 6-12, the simulated pressures for the pressure recorders in the City's Pressure Zone 2 trend closely to the recorded field pressure readings, except for HPR 3. This HPR is located on Dove Street at 24th Avenue, and subsequent evaluation by the City indicates that this hydrant is part of the Oak Lodge Water Distribution System.

Individual graphs of comparisons between model-simulated and field-observed pressures for each HPR are provided in Appendix B. These results indicate that the water demands are properly allocated in the model and that the modeled pipeline network is accurately configured.

6.3.4.3 Pressure Zone 3

Verification results for the City's Pressure Zone 3 indicate that the model-simulated Stanley Tank level for Zone 3 trends well as shown in Figure 6-13. The model-simulated flow at Zone 3 Pump Station trends closely to the SCADA flow. The model results for Pressure Zone 3 indicate that the model was able to closely replicate field conditions.

There were 2 HPRs installed in Pressure Zone 3. As illustrated in Figure 6-14, the simulated pressures for HPR 8 trend closely to the recorded field pressure readings. However, the simulated pressure for HPR 9 was 8 psi higher than the recorded field pressure readings. Based on City staff field investigation on January 4, 2011, the field pressure reading at the location where HPR 9 was installed at 52nd Avenue and King Avenue, ranged from 69.7 to 70.3 psi. These readings are consistent with the model-predicted pressure of 71.2 psi. Therefore, the hydraulic model pressure at the location of HPR 9 is accurate, as predicted in the field. West Yost believes that the pressure discrepancy on HPR 9 during the period it was in the field was most likely caused by an obstruction (*i.e.*, dirt or rock) at the pressure sensor that caused the inaccurate pressure readings. Therefore, we are confident that the model is accurately predicting pressure in Pressure Zone 3. Individual graphs of comparisons between model-simulated and field-observed pressures for each HPR are provided in Appendix B.

6.3.4.4 Pressure Zone 4

Figure 6-15 provides verification results for the City's Pressure Zone 4. Results indicate that the model-simulated flow through the Lava Pump Station trends well, and the model was able to closely replicate field conditions.

As illustrated in Figure 6-15, the simulated pressure for the pressure recorder in the City's Pressure Zone 4 trends closely to the recorded field pressure reading. Individual graphs of comparison between model-simulated and field-observed pressure is provided in Appendix B. These results indicate that the water demands are properly allocated in the model and that the modeled pipeline network is accurately configured.

6.3.5 Hydraulic Model Calibration Findings and Conclusions

In summary, the results from the hydrant tests indicate that the hydraulic model is well calibrated using the preliminary pipeline C-factors assigned as shown in Table 6-1. The model can simulate a fire flow or other demand conditions within the City.

Overall, the results from the verification process validated the system configuration and demand allocation in the hydraulic model except for some minor deviations, which need to be further investigated by the City. Pump station flow rate comparisons at each of City's operated facilities trended well with SCADA recordings. Comparisons of HPR and model-simulated pressure data also trended well. Most of the trends, though not exact, follow closely with the recorded HPR pressures.

Based on the results of the hydraulic model calibration, it can be concluded that the hydraulic model provides a reasonable operational representation of the City's water distribution system and will be a good tool for planning and operational scenarios.

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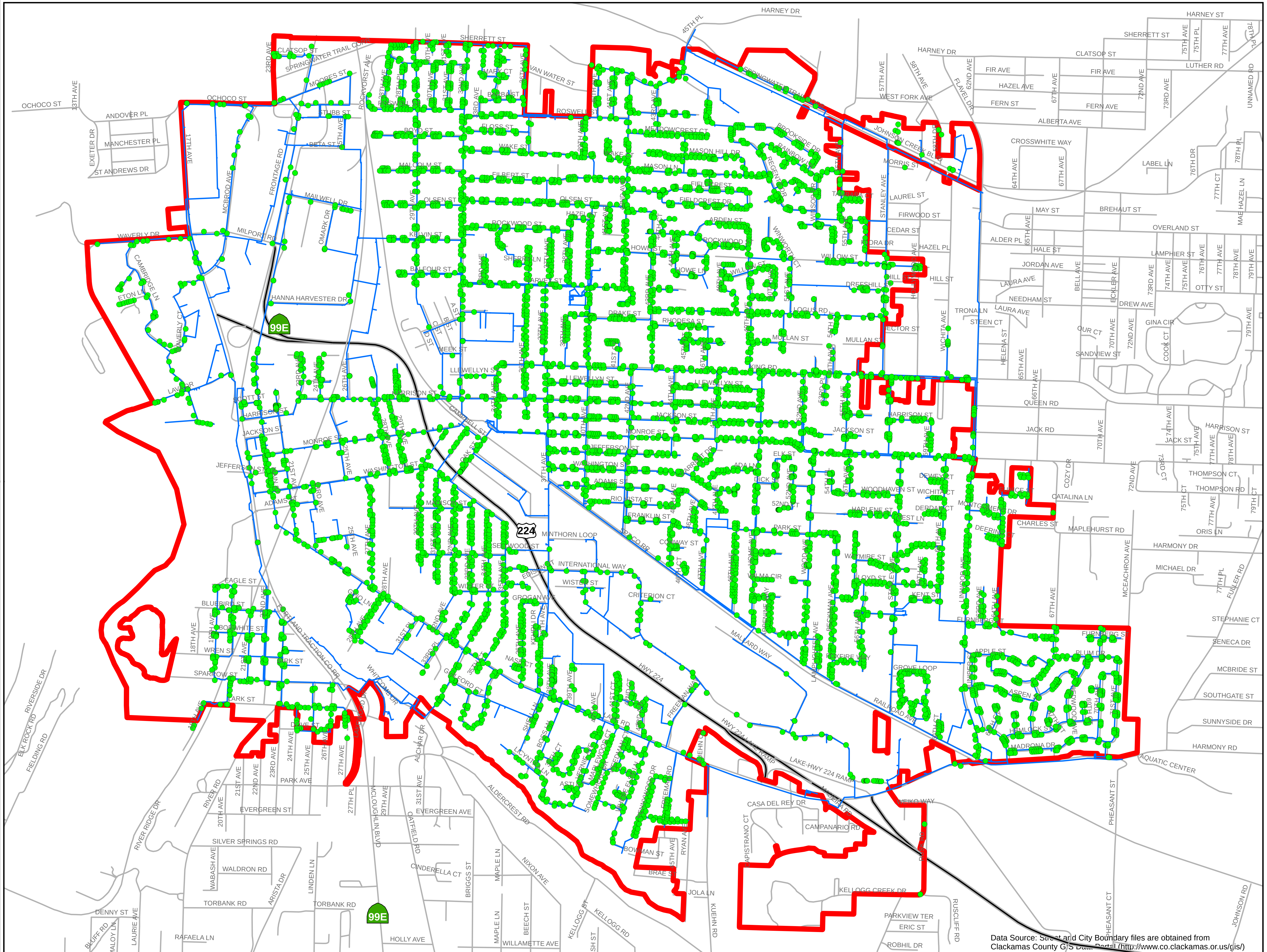
FIGURE 6-2

City of Milwaukie
2010 Water System Master Plan

SPATIALLY LOCATED DEMANDS



0 725 1,450
Scale in Feet

LEGEND
- Spatially Located Demand
- Existing Pipeline
- Service Boundary
- Street
- Highway or Freeway

Data Source: Street and City Boundary files are obtained from Clackamas County GIS Data Portal (http://www.co.clackamas.or.us/gis/)

WEST YOST ASSOCIATES
Consulting Engineers

Figure 6-3. City of Milwaukie Zone 1 Diurnal Pattern
July 10 to 11, 2010

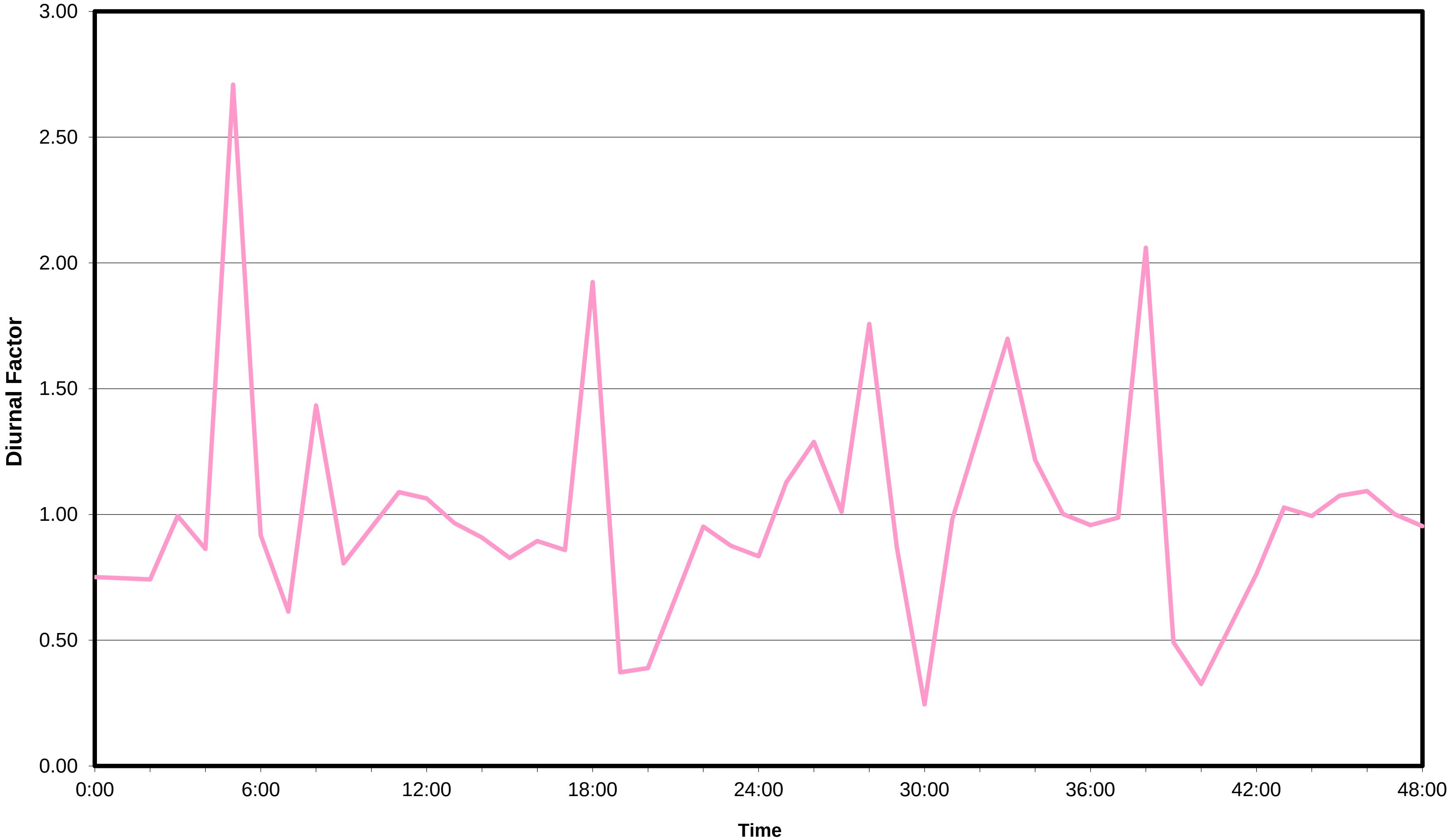


Figure 6-4. City of Milwaukie Zone 2 Diurnal Pattern
July 10 to 11, 2010

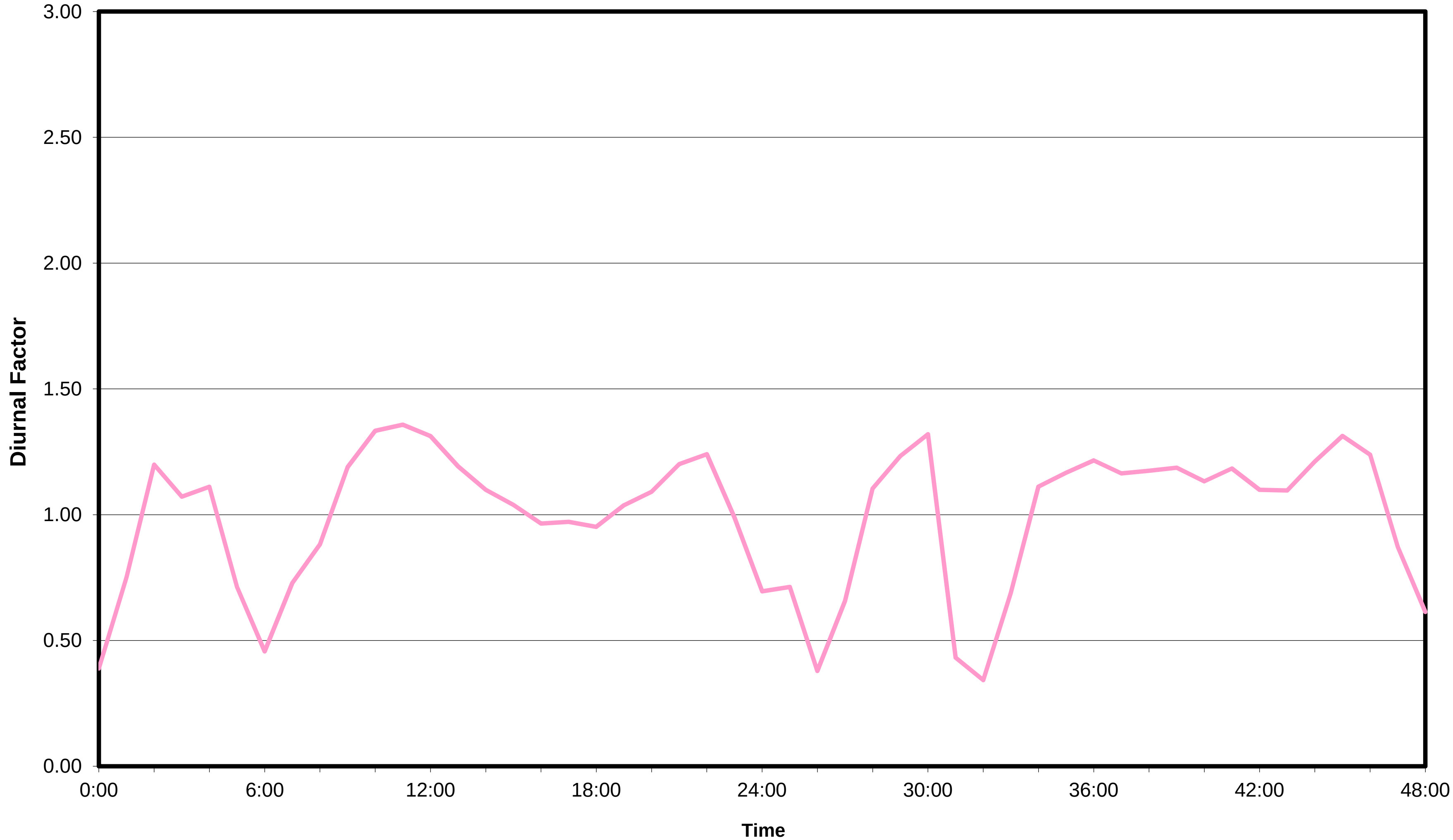


Figure 6-5. City of Milwaukie Zone 3 Diurnal Pattern
July 10 to 11, 2010

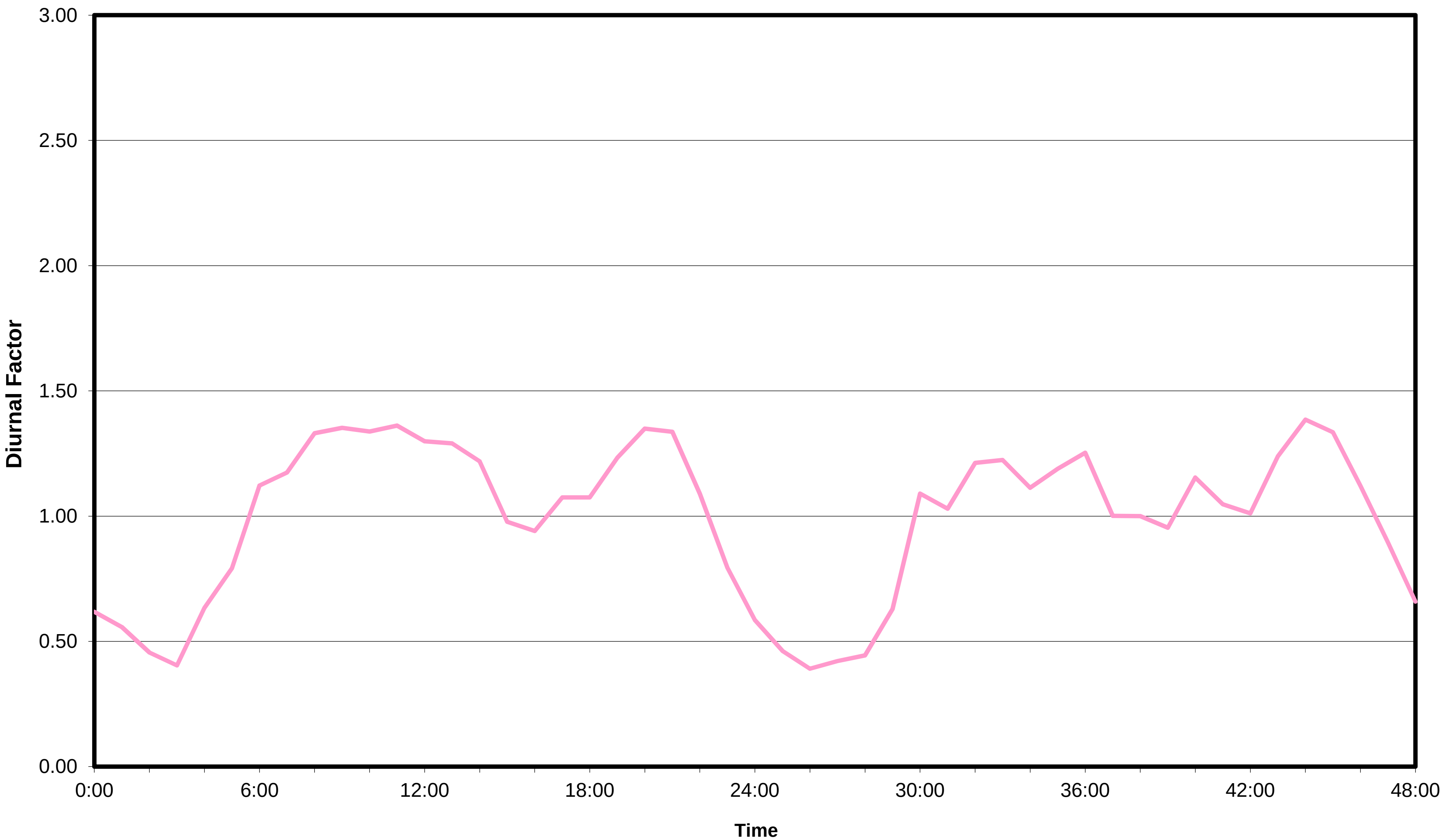


Figure 6-6. City of Milwaukie Zone 4 Diurnal Pattern
July 10 to 11, 2010

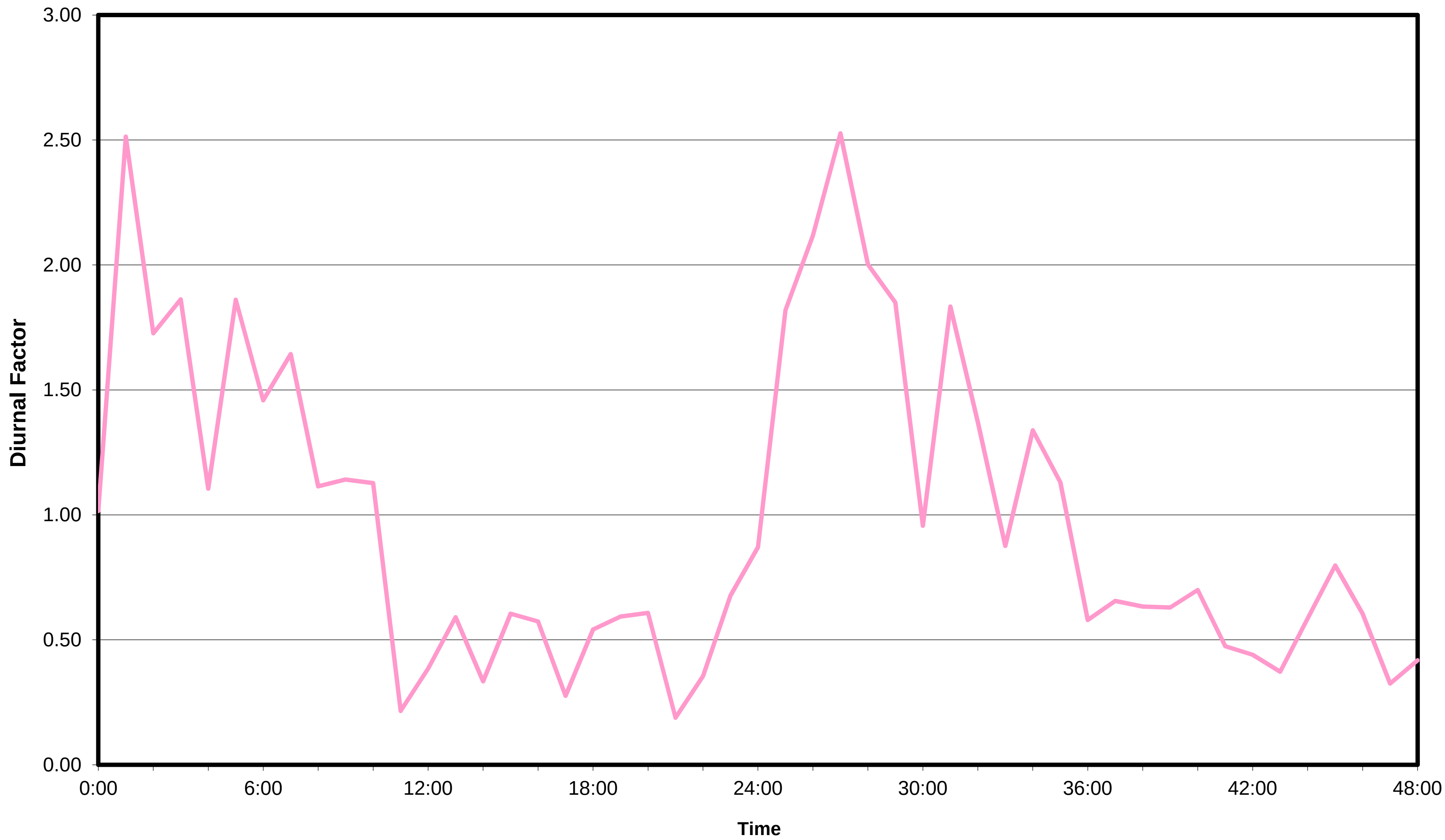


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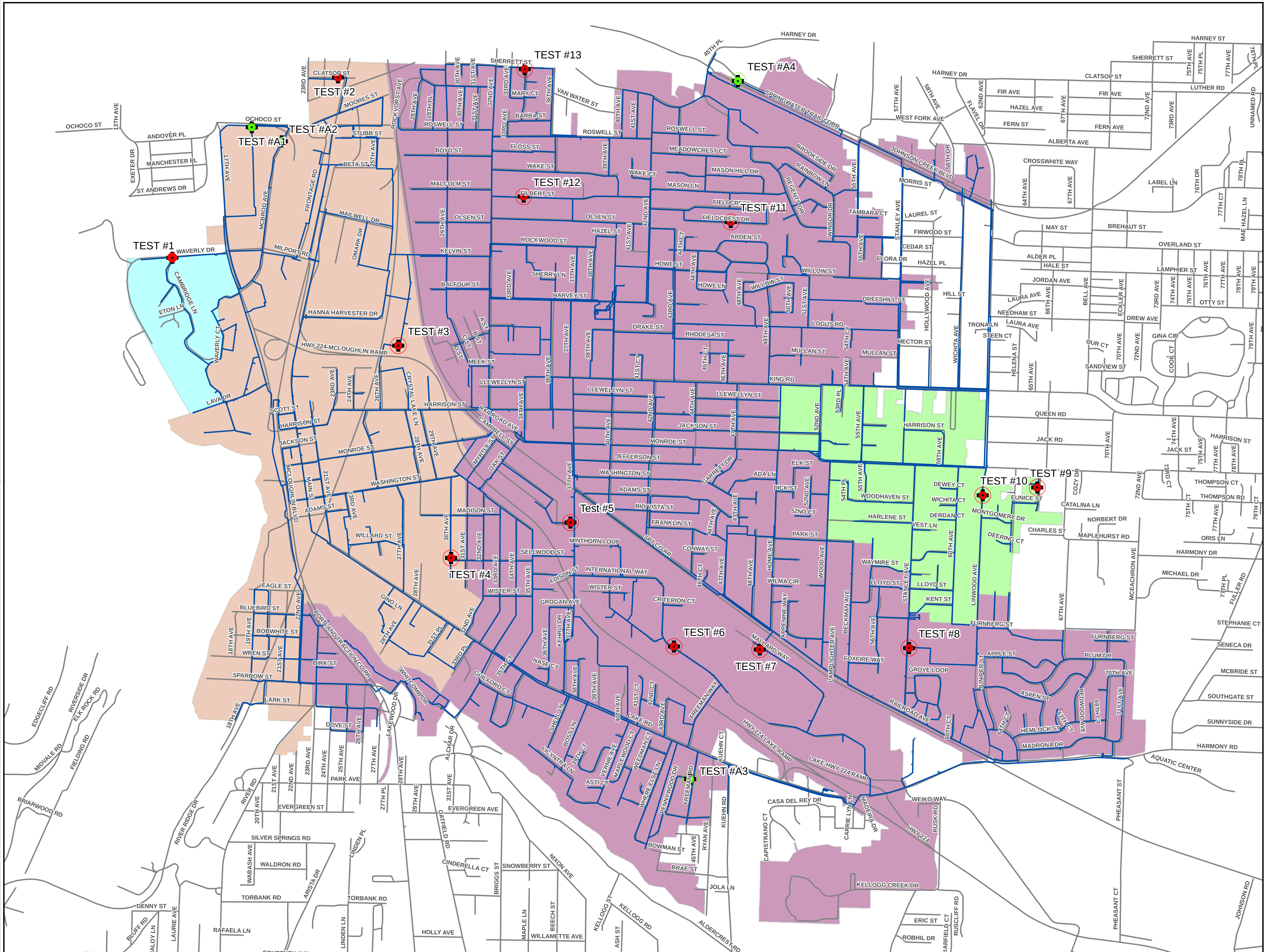
FIGURE 6-7

City of Milwaukie
2010 Water System Master Plan

HYDRANT TEST LOCATIONS



N

0 750 1,500
Scale in Feet

LEGEND

- Test Location - Completed
- Test Location - Canceled
- Alternate Test Location - Completed
- Alternate Test Location - Canceled
- Pipeline
- Zone 1
- Zone 2
- Zone 3
- Zone 4

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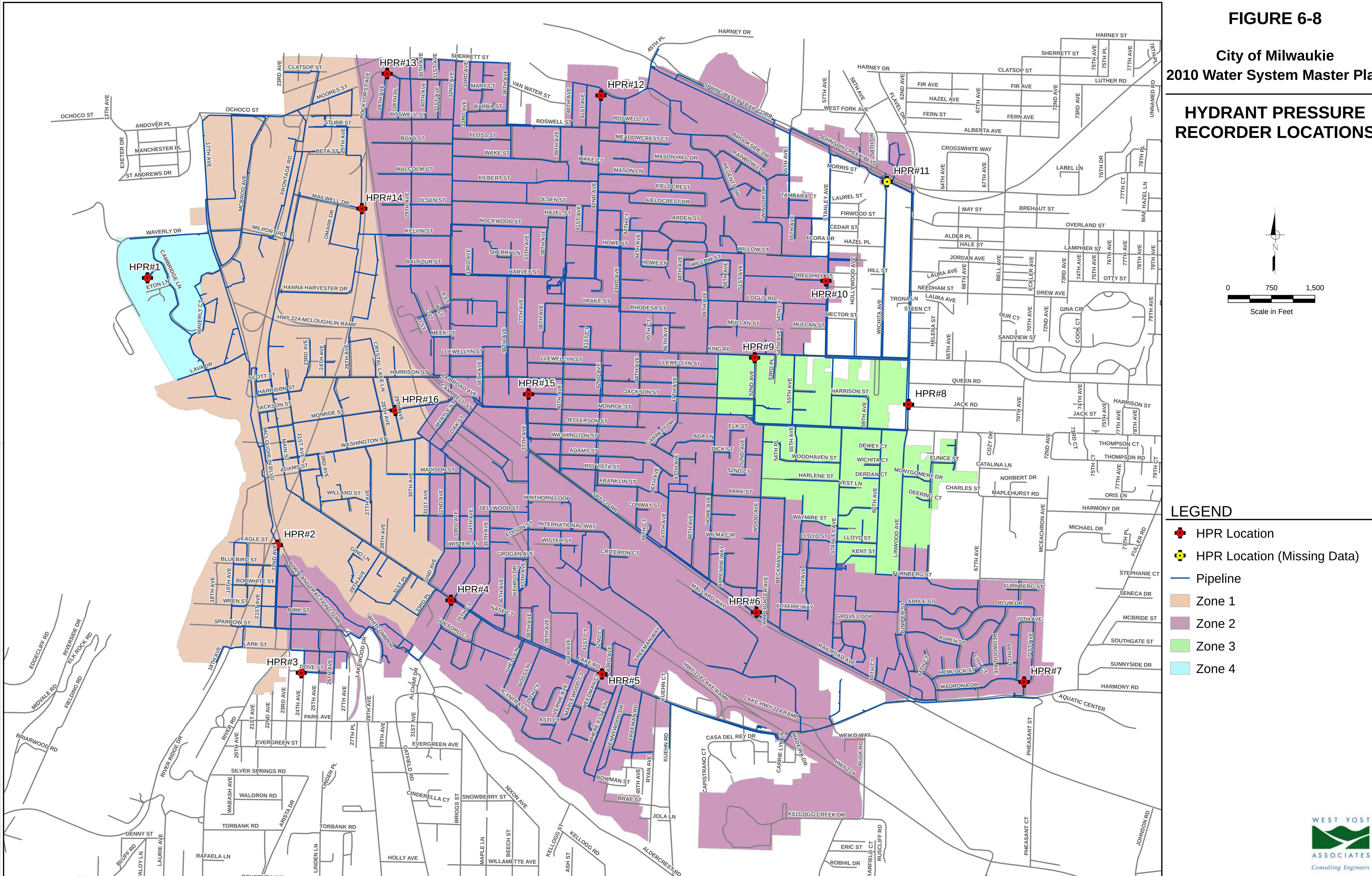

FIGURE 6-8

City of Milwaukie
2010 Water System Master Plan

HYDRANT PRESSURE RECORDER LOCATIONS

LEGEND

- HPR Location
- HPR Location (Missing Data)
- Pipeline
- Zone 1
- Zone 2
- Zone 3
- Zone 4

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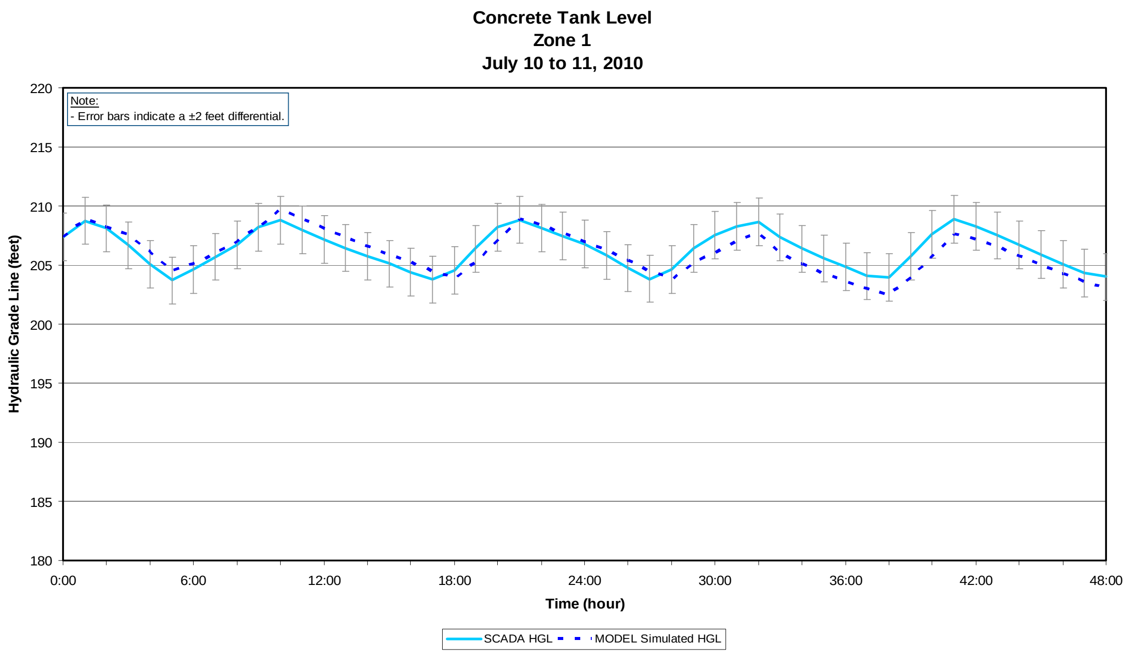
Figure 6-9. Verification for Zone 1



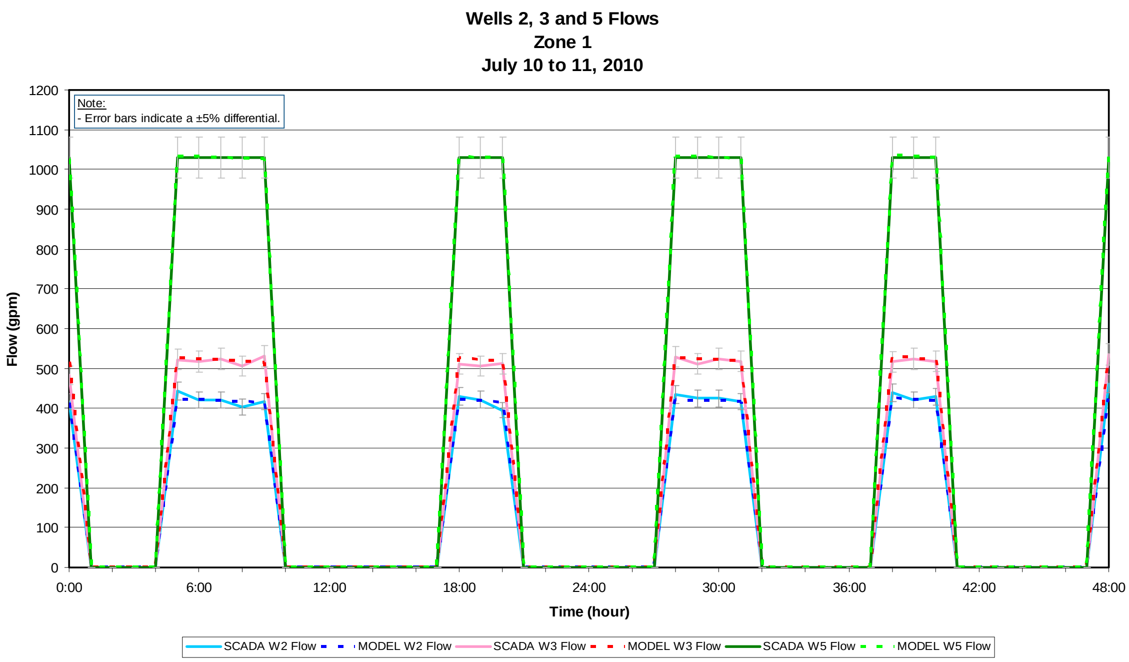
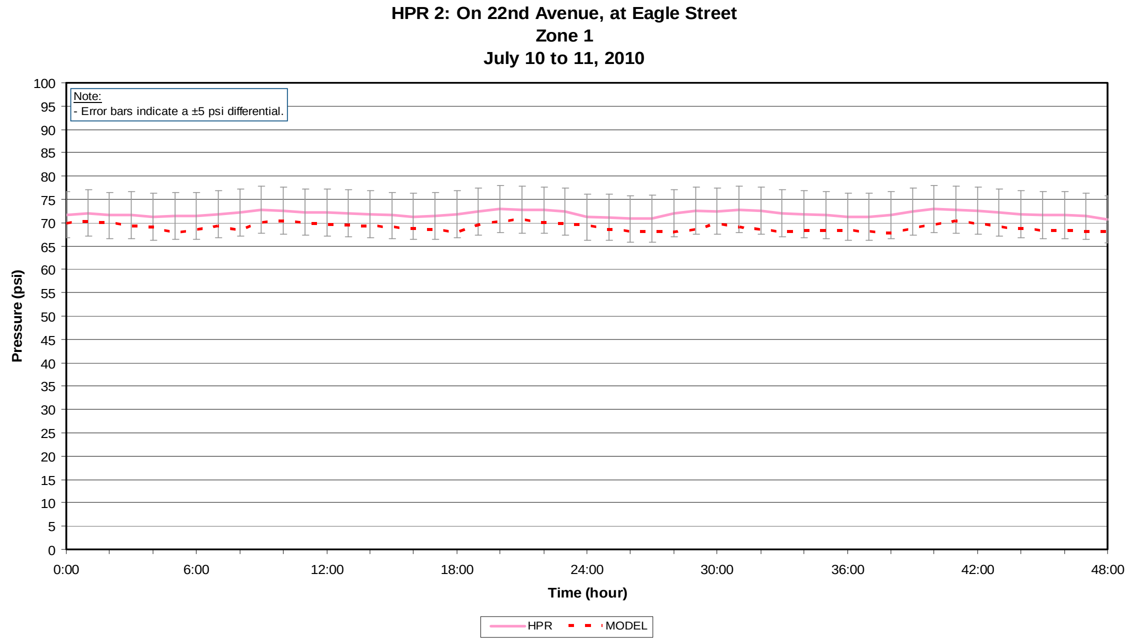


Figure 6-10. Verification for Zone 1



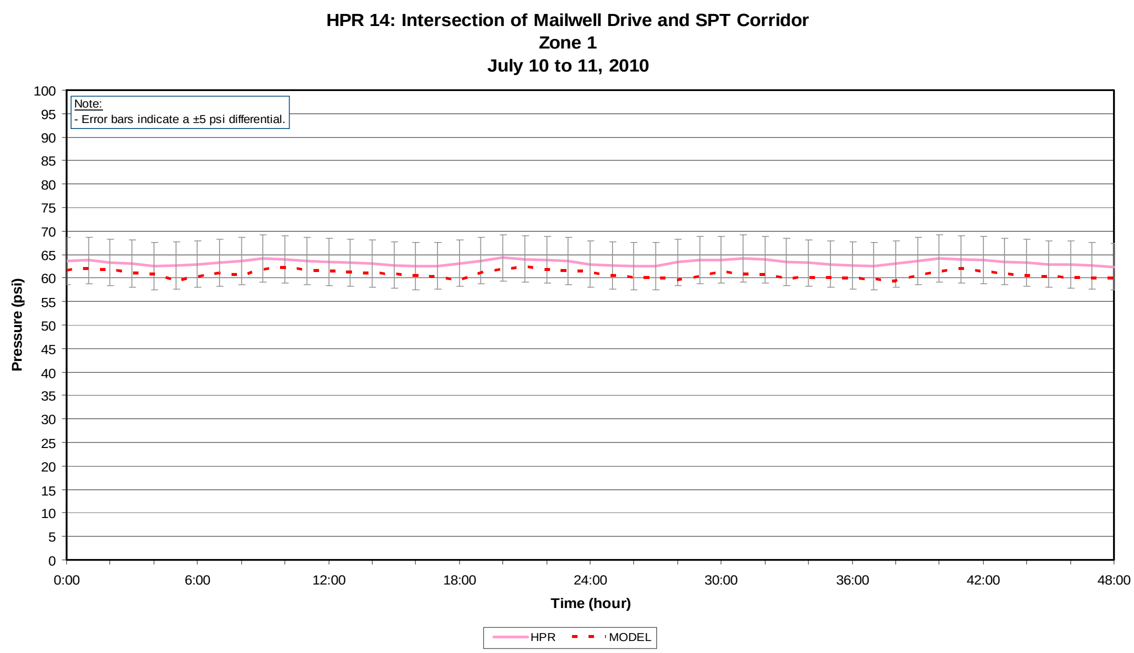
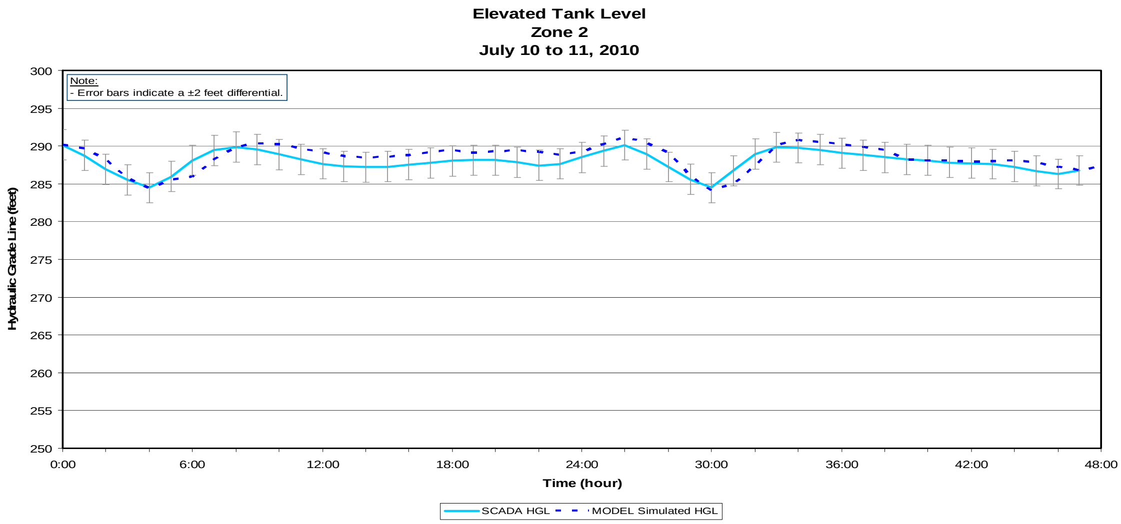
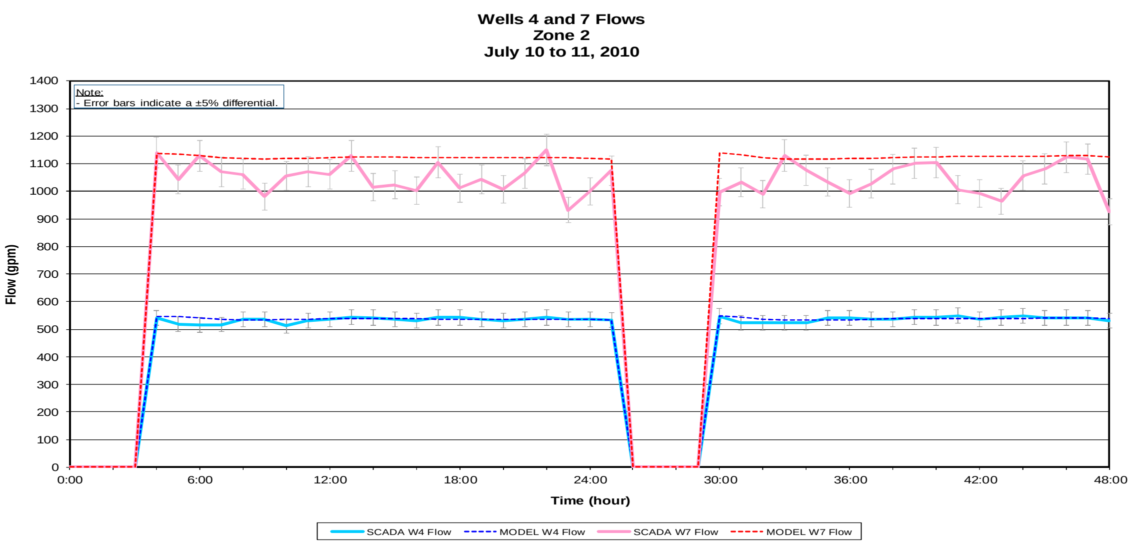


Figure 6-11. Verification for Zone 2





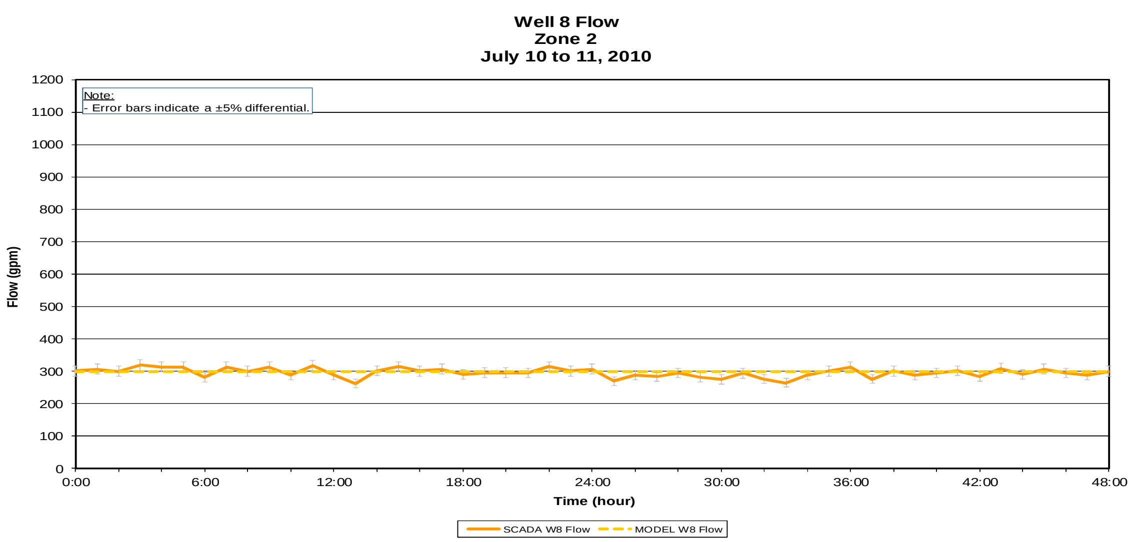
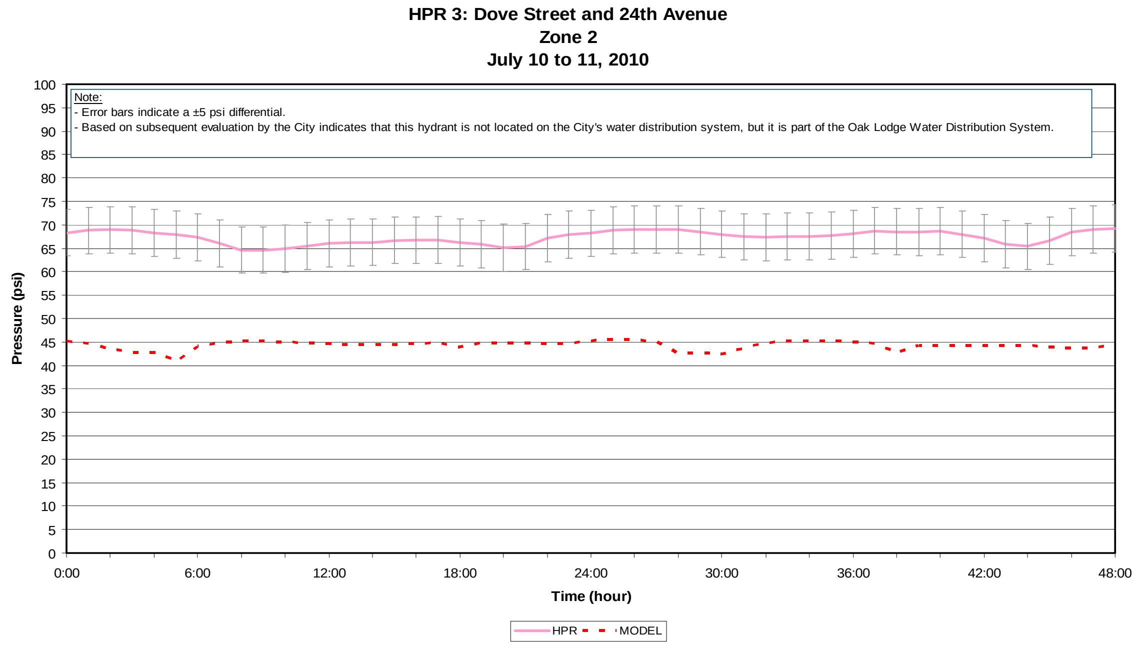


Figure 6-12. Verification for Zone 2



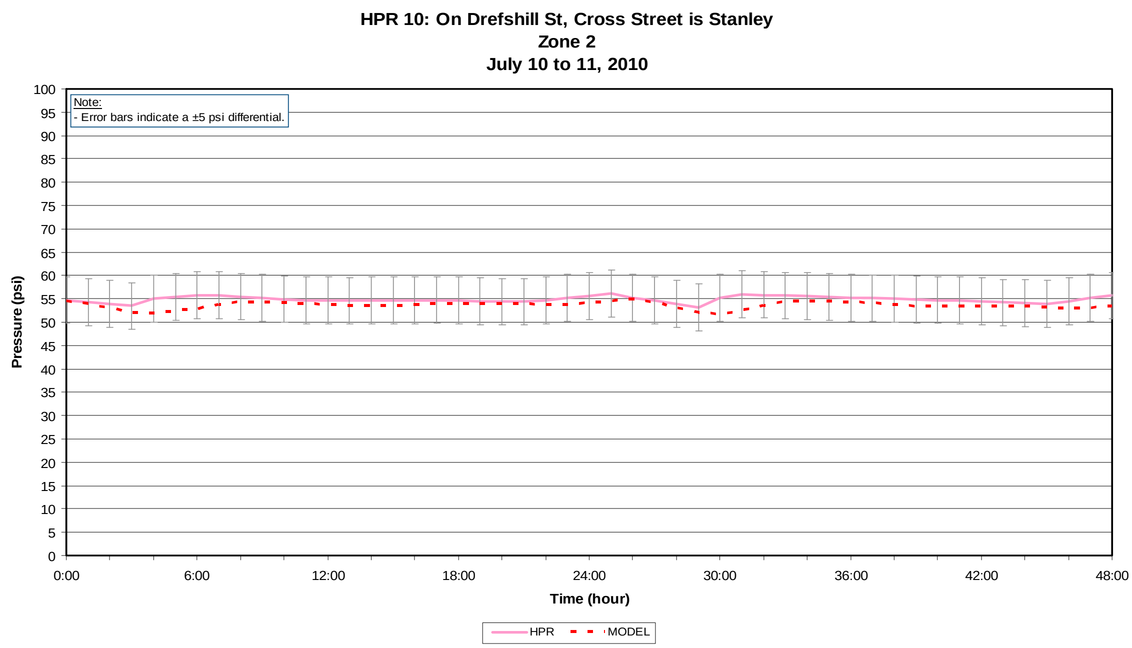
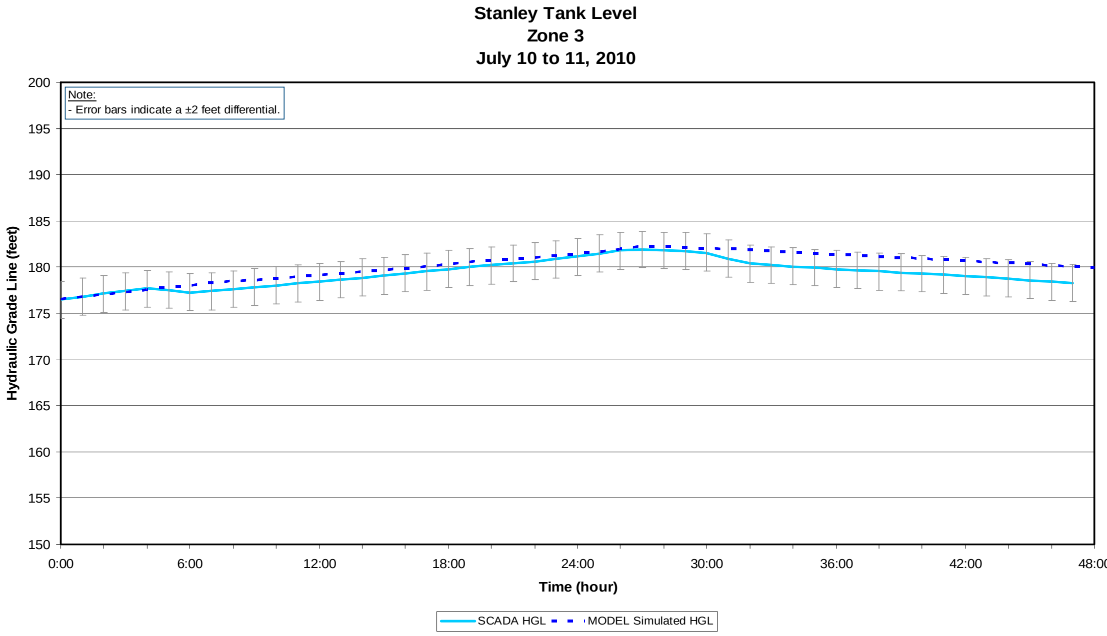


Figure 6-13. Verification for Zone 3



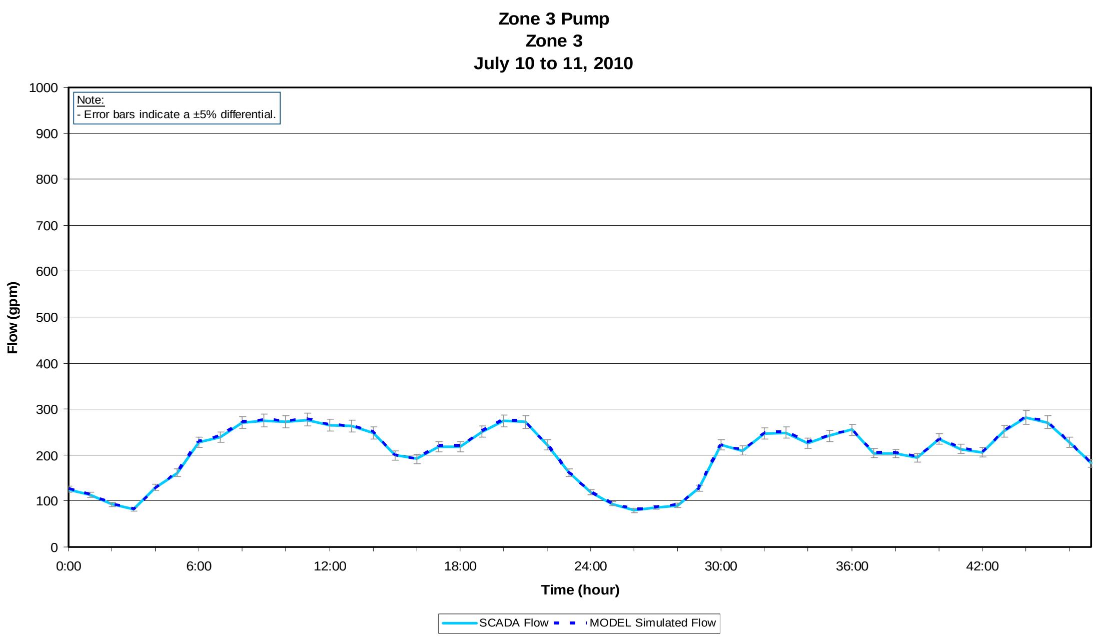
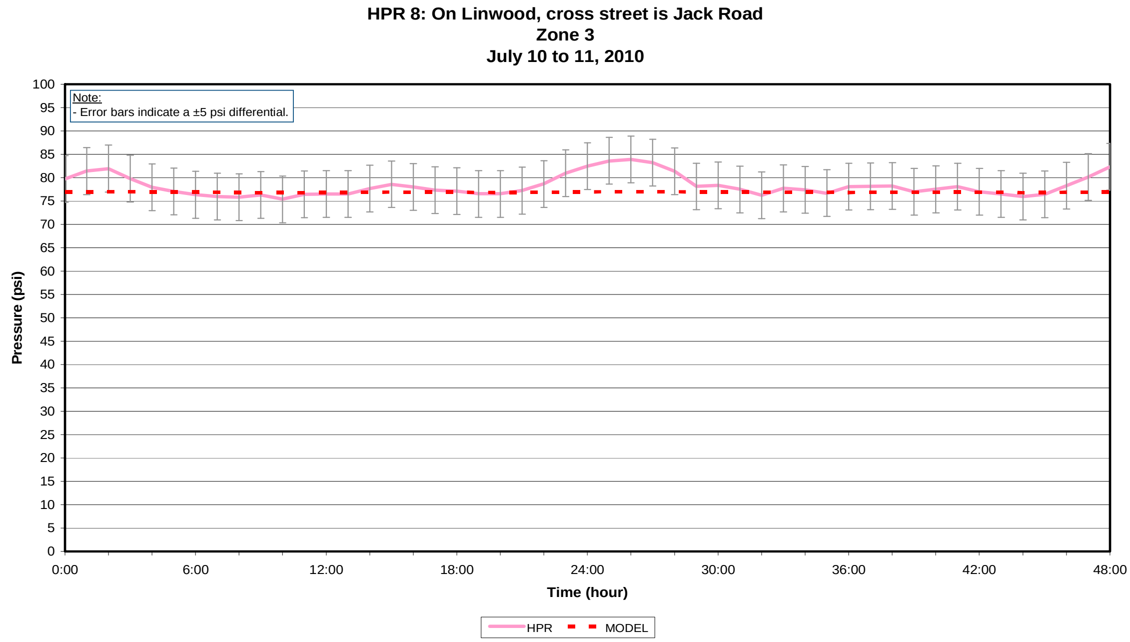


Figure 6-14. Verification for Zone 3



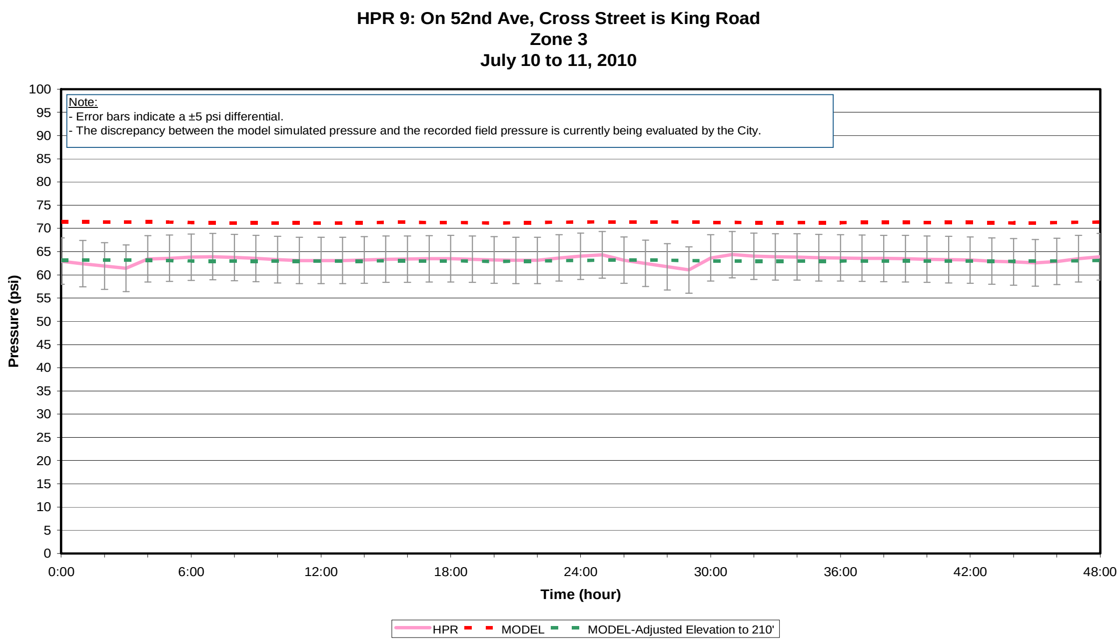
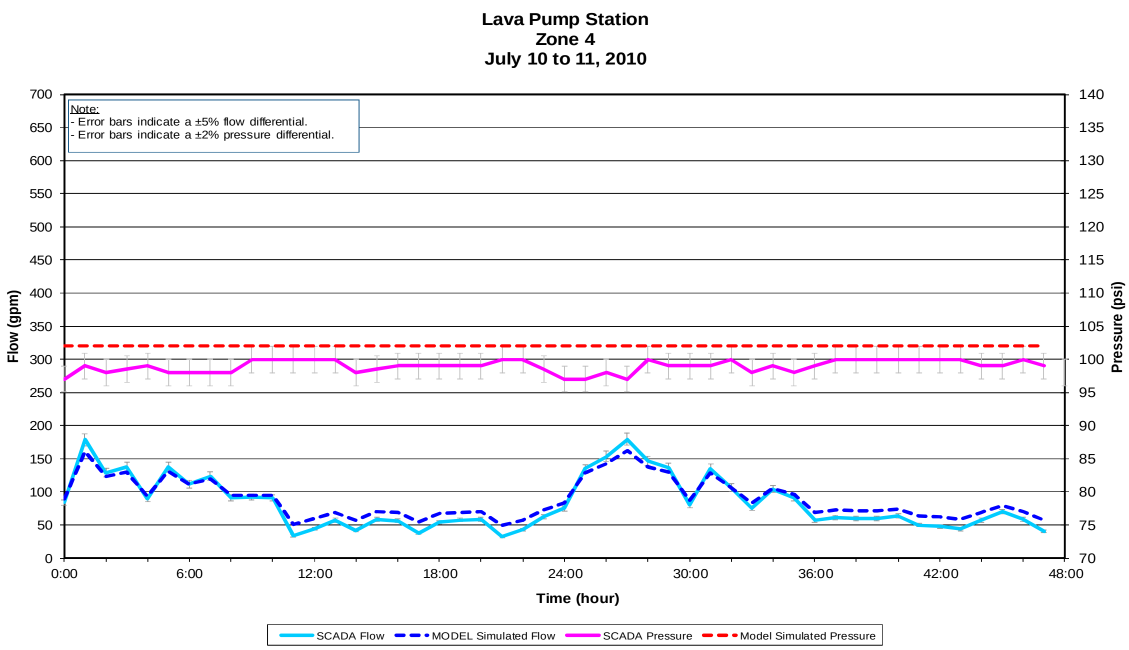
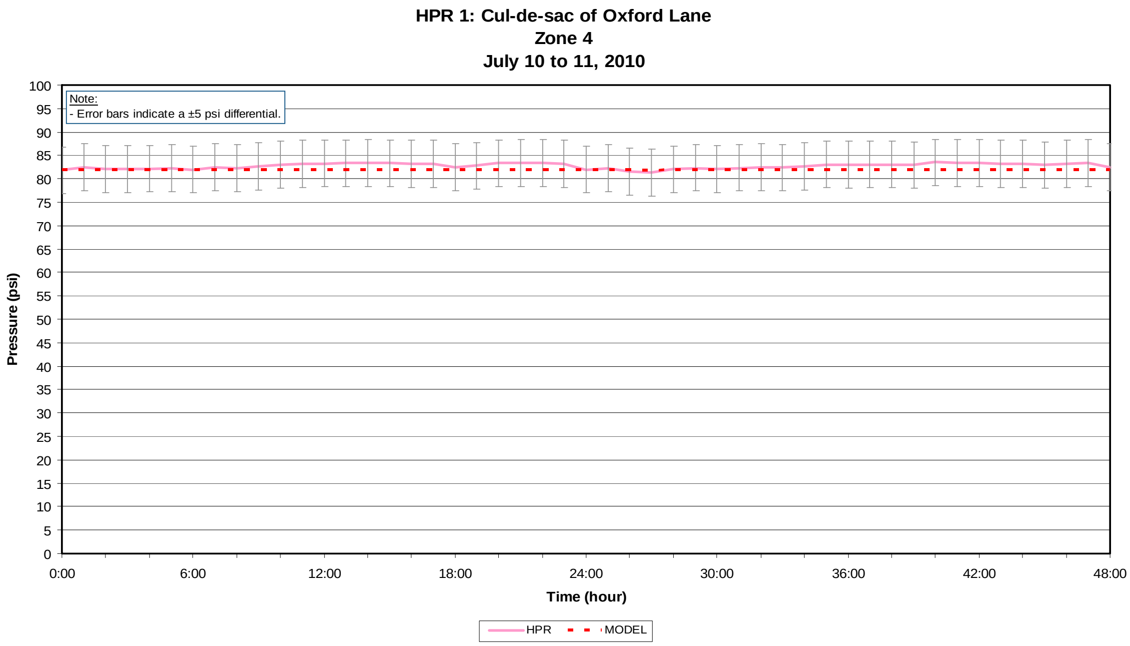


Figure 6-15. Verification for Zone 4





This chapter presents an evaluation of the City's existing water distribution system (see Figure 7-1) and its capability to meet the recommended performance and planning criteria for the City under existing demand conditions. The evaluation includes an analysis of water storage capacity, pumping capacity and the existing distribution system's capacity to meet recommended operational and design criteria. The evaluation was conducted by West Yost using the updated hydraulic model described in Chapter 6. The evaluation, approach, findings and recommendations for addressing the existing water distribution system deficiencies are included and are organized by pressure zone. A description of the recommended CIP to implement the recommended improvements including an estimate of construction costs is provided in Chapter 9.

7.1 EXISTING WATER DEMANDS

The existing water demands for the City's water system are based on production data provided by the City and are presented in Chapter 4. Table 7-1 summarizes the existing demands for each of the City's pressure zones.

Table 7-1. Existing Demands for the City

Pressure Zone	Average Day Demand[a], gpm	Maximum Day Demand[b], gpm	Peak Hour Demand[c], gpm
1	317	602	856
2	1,194	2,268	3,223
3	116	221	314
4	46	87	124
Total	1,673	3,178	4,516

[a] City average day demands are based on 2009 production data.
[b] Maximum day demand is 1.9 times the average day demand.
[c] Peak hour demand is 2.7 times the average day demand.

7.2 WATER STORAGE CAPACITY

7.2.1 Evaluation

There are two ground-level reservoirs and one elevated storage tank in the City's water system. Together, the storage shall be sufficient to meet the City water system's operational, equalization, fire flow, and emergency storage criteria. The volumes required for each of these storage components are presented in Chapter 5 and summarized below:

- Operational Storage: Because the City's treatment plants and booster pumping stations are capable of operating as long as necessary during the maximum demand period, there is no need for dedicated operational storage within the City's distribution system.
- Equalization Storage: 25 percent of maximum day demand;

- Emergency Storage: 100 percent of average day demand; and
- Fire Flow Storage: Per the Clackamas Fire District #1, fire flow storage is equivalent to the maximum fire flow in the pressure zone multiplied by the required duration. For Zones 1, 2 and 3, a fire flow of 2,500 gpm for a duration of 4 hours (Industrial/Commercial) was assumed for this analysis. This fire flow rate is less than the 3,000 gpm listed in Table 7-6 (City Fire Flow Requirements) in that the 500 gpm for fire sprinklers is not included in the storage calculation. For Zone 4, a fire flow of 1,500 gpm for a duration of 3 hours (Multi-Family Residential) was assumed for this analysis.

Because the City's water supply includes wells, the groundwater basin can account for a portion of the recommended water storage and system peaking capacity in the form of a groundwater credit. The emergency storage credit reflects only that groundwater supply which can be reliably accessed when needed (*i.e.*, only wells equipped with auxiliary power). The maximum credit is equal to the required emergency storage capacity.

7.2.2 Results

The existing storage facilities have been evaluated to determine whether they have sufficient capacity to provide the required operational, fire flow, and emergency storage. The City currently has 6.0 MG of water storage as shown in Table 7-2. Analysis of the City's water system indicates that the existing level of water storage is sufficient.

Although it is desirable for each pressure zone to have its own gravity storage, this is not always feasible, especially with small pressure zones or pressure zones that have inadequate elevation for a storage site. In these cases, sharing storage between pressure zones is allowed provided there is a way to convey the required water into the adjacent pressure zone via pressure reducing valves or pump stations. In the case of a pump station, it is also desirable to provide reliable pumping capacity necessary to deliver the storage provided in the adjacent pressure zone.

As shown in Table 7-2, Zone 4 has a storage deficiency of 0.37 MG. However, Zone 4 has access to Zone 1 storage via the Lava Drive Pump Station. Since Pressure Zone 1 has a storage surplus of 0.68, no additional storage is recommended for Zone 4 at this time. However, the Zone 4 booster pump station does not have reliable capacity to pump the required fire flow. Due to space constraints at the Lava Drive Booster Pump Station, serving Zone 4, a dedicated on-site backup generator was not constructed. In lieu of a dedicated on-site generator, a portable generator is stored at the Milwaukie Johnson Creek facility and is dedicated for emergency use only at the Lava Drive Booster Pump Station. Due to the small service area and lack of critical facilities in Zone 4, a dedicated portable generator is acceptable for providing emergency back-up power to convey the fire flow from Zone 1 to Zone 4.

As discussed in Chapter 5, it is important to prevent drainage of reservoirs after a significant earthquake. The recommended improvements are as follows:

- Install a remote controlled shut-off valve or seismic valve on each of the three reservoirs.

Table 7-2. Summary of Existing Storage Requirements

Pressure Zone	Storage Facility	Existing Reservoir Capacity, MG	Required Storage Capacity				Groundwater Storage Credits[d], MG	Total Required Storage, MG	Storage Surplus (Deficiency), MG
			Equalization, MG[a]	Fire Flow, MG[b]	Emergency, MG[c]	Subtotal, MG			
Zone 1	Concrete Reservoir	1.50	0.22	0.60	0.46	1.27	2.17 [d]	0.82	0.68
Zone 2	Elevated Reservoir	1.50	0.82	0.60	1.72	3.14	3.49 [d]	1.42	0.08
Zone 3	Stanley Reservoir	3.00	0.08	0.60	0.17	0.85	0.96 [d]	0.68	2.32
Zone 4	None	0.00	0.03	0.27	0.07	0.37	0.00	0.37	(0.37)
	System Total:	6.00					6.62	3.28	

[a] Based on 25 percent of maximum day demand.

[b] Fire flows based on 4 hr duration x 2,500 gpm sprinklered flow for Industrial/Commercial for Zones 1, 2 and 3 and a 3 hr duration x 1,500 gpm Multi-Family Residential for Zone 4.

[c] Based on 100 percent of average day demand.

[d] Groundwater storage credit is equal to 100% of the total pumping capacity for active wells with backup power. Groundwater storage credit can be used to offset required emergency storage capacity only.

7.3 PUMPING CAPACITY

7.3.1 Ground Water Pumping Capacity

7.3.1.1 Evaluation

The City's pumping capacity was evaluated to assess its ability to deliver a reliable firm capacity to the existing service area. AWWA Manual 31 "Distribution System Requirements for Fire Protection" suggests that a standby pump and reliable power be provided to each pump station. Since a standby pump for groundwater well pumps is not practical, the firm groundwater pumping capacity is defined for the City's groundwater wells as the total well capacity for all wells that can be accessed during a power outage, or all wells with back-up power. All the City's groundwater well pump stations are equipped with backup power; therefore, the groundwater pumping capacity in the City is equal to the total well capacity.

In addition, AWWA suggests that the pumping capacity criterion for the water system be sufficient to meet maximum day demand within the service area, assuming that gravity storage is available. If there is no gravity storage available within the service area, the total pumping capacity must be equivalent to the larger design demand which in the City's case is either maximum day demand plus fire flow or peak hour demand, whichever is greater.

7.3.1.2 Results

The City has gravity storage available within the service area. As a result, the pumping capacity for the water system must be sufficient to meet the maximum day demand. The pumping capacity analysis indicates that the City's existing firm groundwater pumping capacity meets the pumping capacity criterion for the entire service area during a maximum day demand condition. As shown in Table 7-3, the City has a pumping capacity surplus of 1,427 gpm. Table 7-4 summarizes the pumping capacity of each pump station.

Table 7-3. Evaluation of Total Firm Pumping Capacity and Maximum Day Demand

Pump Station	Existing Firm Pumping Capacity, gpm	Existing Maximum Day Demand[a], gpm
Groundwater Wells	4,605[b]	3,178
Pumping Capacity Surplus, gpm	1,427	

(a) Maximum day demand is 1.9 times the average day demand.
(b) Defined as the total active well capacity for all wells that can be accessed during a power outage, or all wells with backup power.

Table 7-4. Summary of Existing Pumping Facilities

Pump Station	Backup Power	Status	Pump 1[a], gpm	Firm Capacity[b], gpm
Well 02	✓	Active	394	394
Well 03	✓	Active	511	511
Well 04	✓	Active	605	605
Well 05	✓	Active	605	605
Well 06	✓	Active	670	670
Well 07	✓	Active	1,120	1,120
Well 08	✓	Active	700	700
			Total	4,605

[a] Pump flow data provided by the City of Milwaukie in Data Request Item 16.
[b] For groundwater well pumps, the firm groundwater pumping capacity is defined as the total well capacity for all wells that can be accessed during a power outage, or all wells with backup power.

7.3.2 Distribution Pumping Capacity

7.3.2.1 Evaluation

The City's water system must be capable of providing the required peak hour demand or maximum day plus fire flow demand to each pressure zone. The distribution pump firm capacity of each pressure zone is defined as the pumping capacity of each pump station serving the pressure zone with the largest pump out of service. If a pump station has a single pump with a back-up generator, the pump capacity is included in the firm capacity. However, if the pressure zone has gravity storage, the required distribution pump firm capacity can be reduced to equal the maximum day demand of the pressure zone. Each pressure zone was analyzed individually taking into consideration that all pressure zones must meet the requirements at the same time. Table 7-5 summarizes the available capacity by pressure zone.

7.3.2.2 Results

Both Zone 1 and Zone 2 have access to gravity storage. As a result, the distribution pump firm capacity must be capable of providing the maximum day demand of the pressure zone. As shown in Table 7-5, the existing firm capacity of Zone 1 exceeds the existing maximum day demand. As a result, the pumping capacity of Zone 1 is sufficient.

The existing firm capacity of Zone 2 is less than the existing maximum day demand. However, the hydraulic model indicates that approximately 668 gpm can be supplied from Zone 1 through the W2 pump station during maximum day demand conditions. With this additional pumping from Zone 1, the resulting pumping capacity of Zone 2 is sufficient.

Both Zone 3 and Zone 4 do not have access to gravity storage. As a result, the distribution pump firm capacity must be equal to the larger of the required peak hour demand or maximum day plus fire flow demand. Table 7-5 indicates that the Zone 3 maximum day plus fire flow demand is greater than the peak hour demand. When the Zone 3 maximum day plus fire flow is compared to the existing firm capacity, the pumping capacity of Zone 3 has an existing deficiency of 1,721 gpm. Adding two 1,750 gpm fire flow pumps to the Zone 3 pump station would resolve this deficiency.

The existing firm capacity of Zone 4 exceeds the existing maximum day plus fire flow demand and the existing peak hour demand. As a result, the pumping capacity of Zone 4 is sufficient.

7.3.3 Critical Pumping Facilities

All critical pumping facilities should be equipped with an on-site, stand-by power generator. Critical pumping facilities are defined as those facilities that provide service to pressure zones and/or service areas without sufficient emergency storage and that meet the following criteria:

- The largest facility that provides water to a particular pressure zone and/or service area;
- A facility that provides the sole source of water to single or multiple pressure zones and/or service area(s);
- A facility that provides water from key groundwater supply wells (depends on capacity, quality, and location) into a pressure zone and /or service area.

As indicated in Table 7-4 and Table 7-5, all wells, pump stations and treatment facilities are equipped with back-up generators able to provide pumping capacity during a power outage. Due to space constraints at the Lava Drive Booster Pump Station, serving Zone 4, a dedicated on-site back-up generator was not constructed. A portable generator is stored at the Milwaukie Johnson Creek facility and is dedicated for emergency use only at the Lava Drive Booster Pump Station.

Table 7-5. Summary of Pumping Capacity Requirements

Service Area	Pump Station	Total Capacity, gpm	Firm Capacity, gpm[(a)]	Existing Backup Power	Existing Average Day Demand	Existing Maximum Day Demand[(b)]	Required Fire Flow, gpm	Existing Maximum Day Demand plus Fire Flow	Existing Peak Hour Demand[(c)]	Required Additional Firm Pumping Capacity, gpm
Zone 1	Treatment Plant 235 No. 1	700	1,400	Diesel Generator	317	602	2,500	3,102	856	0[(d)]
	Treatment Plant 235 No. 2	700		Diesel Generator						
	Treatment Plant 235 No. 3	700		Diesel Generator						
	Subtotal	2,100	1,400							
Zone 2	Treatment Plant 47 No. 1	900	900	Diesel Generator	1,194	2,268	2,500	4,768	3,223	0[(e)]
	Treatment Plant 47 No. 2	900		Diesel Generator						
	Well 8	700	700	Diesel Generator						
	Subtotal	2,500	1,600							
Zone 3	Zone 3 Pump No. 1	200	1,000	Diesel Generator	116	221	2,500	2,721	314	1,721[(f)]
	Zone 3 Pump No. 2	200		Diesel Generator						
	Zone 3 Pump No. 3	600		Diesel Generator						
	Zone 3 Pump No. 4	600		Diesel Generator						
	Subtotal	1,600	1,000							
Zone 4	Lava Pump 1	300	3,800	Portable Diesel Generator	46	87	1,500	1,587	124	0[(g)]
	Lava Pump 2	300		Portable Diesel Generator						
	Lava Pump 3	1,750		Portable Diesel Generator						
	Lava Pump 4	1,750		Portable Diesel Generator						
	Subtotal	4,100	3,800							
	Total Pumping Capacity, gpm	10,300	7,800		1,673	3,178		12,178	4,516	1,721
	Total Pumping Capacity, mgd	14.8	11.2		2.4	4.6		17.5	6.5	2.5

(a) Firm capacity for booster pump stations is defined as pumping capacity with the largest pump out of service and/or single pump with backup generator. Available groundwater pumping capacity is defined as the pumping capacity of all wells that can be accessed during a power outage, or all wells with backup power.

(b) Maximum day demand equals 1.9 times average day demand.

(c) Peak hour demand equals 2.7 times average day demand.

(d) To provide a more conservative estimate of pumping capacity, the Treatment Plant 235 pumps supplying Zone 1 were used in this calculation. The available pumping capacity of the wells serving this zone is equal to 1,510 gpm. Since this zone has access to gravity storage, the pumping capacity is required to be equal to the maximum day demand for the pressure zone.

(e) To provide a more conservative estimate of pumping capacity, the Treatment Plant 47 pumps supplying Zone 2 as well as well 8 were used in this calculation. The available pumping capacity of the wells serving this zone is equal to 2,425 gpm. Since this zone has access to gravity storage, the pumping capacity is required to be equal to the maximum day demand for the pressure zone. The hydraulic model indicates that approximately 668 gpm can be supplied from Zone 1 through the W2 pump station during maximum day demand conditions. Based on this, the additional pumping capacity required is Maximum Day Demand (2,268 gpm) – Firm Pumping Capacity (1,600 gpm) – W2 pump station (668 gpm) = 0 gpm.

(f) Because all water supplied to this zone is pumped from the Stanley Reservoir by the Zone 3 booster pumps, the pumping capacity of the booster pump station was used in this calculation. Because this zone does not have access to gravity storage, the pumping capacity for the pressure zone is required to be equal to the maximum day demand plus fire flow or peak hour demand, whichever is greater.

(g) Because all water supplied to this zone is pumped from Zone 1, the pumping capacity of Lava Drive Pump Station was used in this calculation. Because this zone does not have access to gravity storage, the pumping capacity for the pressure zone is required to be equal to the maximum day demand plus fire flow or peak hour demand, whichever is greater.

(h) Highlighted cells refer to the demand condition that applies to each pressure zone.

7.4 WATER DISTRIBUTION SYSTEM

7.4.1 Methodology

A steady state hydraulic analysis using the City's updated and calibrated hydraulic model (discussed in Chapter 6), was conducted to identify areas of the existing water distribution system that do not meet the recommended system performance criteria. The results of the evaluation of the water distribution system are presented below for the following existing demand scenarios:

- Peak Hour Demand—peak hour demands are met by either flows from the storage reservoirs or from the pump stations. A peak hour flow condition was simulated for the existing distribution facilities to evaluate their capability to meet this peak hour demand condition.
- Maximum Day Demand plus Fire Flow—to evaluate the system under the maximum day demand plus fire flow condition, a two-step analysis was performed. The first step used the InfoWater's "Available Fire Flow Analysis" option to determine if the minimum pressure and required fire flow could be met with the existing water facilities. Fire flow demands were assigned by land use type and simulated at existing hydrant locations in each pressure zone. If the analysis indicated that the system failed to meet the minimum requirements for pressure and flow, a second analysis was performed. The second analysis involved running the model with pipeline improvements/system modifications added to the distribution system to eliminate previously identified deficiencies.

As shown in Table 7-1, the City's existing service area peak hour demand was calculated to be 4,516 gpm (6.5 mgd). This peak hour demand represents a peaking factor of 2.7 times the average day demand. During a peak hour condition, a minimum pressure of 40 psi must be maintained throughout the system. Maximum head loss per thousand feet of distribution main generally should not exceed 10 ft/kft and maximum velocities should not exceed 7 fps. Details of system pressures in each of the City's pressure zones as simulated in the model under the peak hour demand condition are discussed below.

Fire flow demands were simulated at various locations within the City's service area to determine whether or not the minimum residual pressure criterion of 20 psi within the pressure zone could be maintained during a maximum day demand condition with a concurrent fire flow. Table 7-6 presents fire flow demand requirements based on land use categories (illustrated in Figure 7-2). This fire flow data is also illustrated on Figure 7-3.

Table 7-6. City Fire Flow Demand Requirements[a]

Land Use	Fire Flow, gpm	Duration, hours
Single-Family Residential[b]	1,500	2
Multi-Family Residential	1,500	3
Institutional[c]	2,000[e]	4
Industrial/Commercial[d]	3,000[e]	4

(a) Specific fire flows were determined from Table B105.1 of the 2007 OFC, and depend on construction type and fire area. These fire flow requirements are based on Institutional and Industrial/Commercial buildings being fully sprinkled.
(b) Single-Family includes Low Density Residential, Medium Density Residential and Moderate Density Residential land use.
(c) Institutional includes Public and Town Center land uses.
(d) Industrial/Commercial includes Commercial, Mixed Use and, Industrial land uses.
(e) Includes a 500 gpm demand for on-site sprinkler flow.

Pipelines are typically designed to deliver peak hour flows and maximum day demands plus fire flows within acceptable pressure, velocity, and head loss ranges as stated in Chapter 6.

7.4.2 EVALUATION & RESULTS

The following addresses the results of the peak hour demand and maximum day demand plus fire flow analyses by individual pressure zone.

7.4.2.1 Pressure Zone 1

7.4.2.1.1 Peak Hour

During a peak hour demand condition, results indicate that the existing system in Pressure Zone 1 can adequately deliver peak hour demands to most of the Zone under the City's minimum pressure criteria of 40 psi (see Figure 7-4). System pressures in the zone range from 29 to 74 psi. There are two general areas within Zone 1 with pressures below 40 psi. These areas are either within a couple of psi of the acceptable range, or are located above the elevation that will support a 40 psi pressure given the HGL of Pressure Zone 1.

- Lake Road (East of 32nd Avenue): Peak hour pressures in this area range from 38 to 39 psi. Since this pressure is so close to the City minimum requirement, no mitigation is recommended.
- Sparrow Street & River Road: Peak hour pressures along River Road near Wren Street range from 29 to 38 psi. Locations near the intersection of Sparrow Road and River Road are situated at an elevation of 133.5 feet, which is above the service elevation of Zone 1. Because this area is near the Zone 2 boundary, and the pressure deficiency is not related to pipeline configuration, West Yost recommends the City adjust the pressure zone boundary to include this area in the Zone 2 service area (see Figure 7-5).

As illustrated on Figure 7-4, the majority of the pipelines in Pressure Zone 1 meet the City's head loss criteria of 10 ft/kft. There were several short segments of pipeline that exceeded the recommended head loss criteria; however after discussions with the City, it was agreed that these segments are located at non-critical locations, thus no mitigation is recommended at this time.

7.4.2.1.2 Maximum Day plus Fire Flow Demand

Before running the maximum day plus fire flow demand analysis, all 4-inch diameter pipelines in Zone 1 were upsized to 8-inch diameter pipelines. Forty three pipelines comprising 1,894 linear feet of pipeline were upsized.

Fire flows were simulated at hydrant locations throughout Zone 1 based on land use. Figure 7-6 represents the residual pressures at Zone 1 hydrants during maximum day plus fire flow conditions. The fire flow simulation results show that many areas in Pressure Zone 1 cannot maintain a minimum system pressure of 20 psi under the required fire flow. Figure 7-7 presents the available fire flow at a 20 psi residual throughout Pressure Zone 1 system. Forty four (44) hydrants could not meet the minimum flow requirements for each of the associated land uses. This represents approximately 27 percent of the modeled hydrants within the pressure zone.

Approximately 12 percent of the modeled hydrants in Pressure Zone 1 were unable to provide even the minimum fire flow of 1,500 gpm. However, when the fire flow demand of 1,500 gpm was split between two hydrants, the required residual pressure of 20 psi could be met. These locations are illustrated in red on Figure 7-7.

The required upgrades to the existing system for Zone 1 are extensive, and completion of pipeline upgrades for the sole purpose of improving fire flow would be cost prohibitive to the existing customers of the City. Therefore, it is not recommended that all of the improvements necessary to increase fire flow capacity in the existing water system be identified as projects in the City's Capital Improvement Program (CIP). Instead, the improvement projects have been prioritized based on the existing water system's size, age, and proximity to public facilities. The highest priority projects are included in the CIP and consist of correcting existing fire flow deficiencies in areas zoned "public", replacing existing 4-inch diameter water mains constructed prior to 1960, and replacing existing 6-inch diameter water mains constructed prior to 1960.

Figure 7-8 illustrates the location of 4-inch and 6-inch diameter pipelines that were constructed prior to 1960. The total length of the 4-inch diameter pipeline prior to 1960 is 10 feet. The total length of the 6-inch diameter pipeline prior to 1960 is 15,126 lineal feet. The maximum day plus fire flow demand analysis was rerun after upsizing these 4-inch and 6-inch diameter pipelines to 8-inch diameter pipelines. Figure 7-9 illustrates the residual pressures within Zone 1 under these pipeline improvements. As illustrated in Figure 7-9, there are few less areas that could not meet the minimum required fire flow while maintaining the minimum 20 psi residual pressure. West Yost recommends the City improve the fire flow requirement in these areas once the capital improvement program is in place in the future. Figure 7-10 presents the available fire flow at a 20 psi residual during maximum day plus fire flow demand condition with the upsizing of 4-inch and 6-inch diameter pipelines prior to 1960 to 8-inch diameter pipelines.

Hydrants serving areas zoned "public" were evaluated to insure they could meet the required fire flow of 2,000 gpm (Figure 7-7). One hydrant in Zone 1 was unable to meet this requirement. The recommended improvements are as follows:

- Upsize approximately 320 feet of 6-inch diameter pipeline to 8-inch diameter pipeline from the hydrant location to Willard Street. See Figure 7-11.

7.4.2.2 Pressure Zone 2

7.4.2.2.1 Peak Hour

During a peak hour demand condition, results indicate that the existing system in Pressure Zone 2 is adequate to deliver peak hour demands under the City's minimum pressure criteria of 40 psi (see Figure 7-12). Pressures within Pressure Zone 2 ranged from 40 to 114 psi. Those locations with pressures greater than 100 psi are located below the minimum service elevation of Zone 2. This includes SE Whitcomb Avenue between Short Street and Oatfield Road.

As illustrated on Figure 7-12, the majority of the pipelines in Pressure Zone 2 meet the City's head loss criteria of 10 ft/kft. There were several short segments of pipeline that exceeded the recommended head loss criteria; however, after discussions with the City, it was agreed that these segments are located at non-critical locations, thus no mitigation is recommended at this time.

7.4.2.2.2 Maximum Day plus Fire Flow Demand

Before running the maximum day plus fire flow demand analysis, all 4-inch diameter pipelines in Zone 2 were upsized to 8-inch diameter pipelines; 296 pipelines comprising 28,358 linear feet of pipeline were upsized.

Fire flows were simulated at hydrant locations throughout Zone 2 based on land use. Figure 7-13 represents the residual pressures at Zone 2 hydrants during maximum day plus fire flow conditions. The fire flow simulation results show that many areas in Pressure Zone 2 cannot maintain a minimum system pressure of 20 psi under the required fire flow. Figure 7-14 presents the available fire flow at a 20 psi residual throughout Pressure Zone 2. Thirty-eight (38) hydrants of the 504 hydrants modeled could not meet the minimum flow requirements for each of the associated land uses. This represents approximately eight percent of the hydrants within the pressure zone. Approximately six percent of the modeled hydrants in Pressure Zone 2 were unable to provide even the minimum fire flow of 1,500 gpm. However, when the 1,500 gpm fire flow demand was split between two hydrants, the required residual pressure of 20 psi could be met. These locations are illustrated in red on Figure 7-14.

The required upgrades to the existing system for Zone 2 are extensive, and completion of pipeline upgrades for the sole purpose of improving fire flow would be cost prohibitive to the existing customers of the City. Therefore, it is not recommended that all the improvements necessary to increase fire flow capacity in the existing water system be identified as projects in the City's CIP. Instead, the improvement projects have been prioritized based on the existing water system's size, age, and proximity to public facilities. The highest priority projects are included in the CIP and consist of correcting existing fire flow deficiencies in areas zoned "public", replacing existing 4-inch diameter water mains constructed prior to 1960, and replacing existing 6-inch diameter water mains constructed prior to 1960. Figure 7-8 illustrates the location of any existing 6-inch diameter water mains constructed prior to 1960. The total length of the 6-inch diameter pipeline prior to 1960 is 49,373 lineal feet. The total length of the 4-inch diameter pipeline prior to 1960 is 10,582 lineal feet. These 4-inch and 6-inch diameter pipelines were upsized to 8-inch diameter pipelines, and reran in the hydraulic model. Figure 7-15 presents the residual pressures within Zone 2 under these pipeline improvements, and Figure 7-16 presents the available fire flow at 20 psi residual pressure.

Hydrants serving areas zoned "public" were evaluated to insure they could meet the required fire flow of 2,000 gpm (Figure 7-14). Two hydrants in Zone 2 were unable to meet this requirement. The recommended improvements are as follows:

- Upsize approximately 600 feet of 6-inch diameter pipeline to 8-inch diameter pipeline and upsize approximately 95 feet of 4-inch diameter pipeline to 8-inch diameter pipeline in the area to the west of Flavel Drive (see Figure 7-17).

7.4.2.3 Pressure Zone 3

7.4.2.3.1 Peak Hour

During a peak hour demand condition, results indicate that the existing system in Pressure Zone 3 could adequately deliver peak hour demands under the City's minimum pressure criteria of 40 psi (see Figure 7-18). System pressures within the pressure zone ranged from 65 to 88 psi.

Head losses and velocities in Pressure Zone 3 meet the City design criteria of not more than 10 ft/kft and 7 fps, respectively.

7.4.2.3.2 Maximum Day plus Fire Flow Demand

Before running a maximum day plus fire flow demand analysis, all 4-inch diameter pipelines in Zone 3 were upsized to 8-inch diameter pipelines. Forty-five pipelines comprising 6,683 linear feet of pipeline were upsized.

Fire flows were simulated at hydrant locations throughout Zone 3 based on land use. Figure 7-19 represents the residual pressures at Zone 3 hydrants during maximum day plus fire flow conditions. The fire flow simulation results show that many areas in Pressure Zone 3 cannot maintain a minimum system pressure of 20 psi under the required fire flow.

Figure 7-20 presents the available fire flow at a 20 psi residual throughout the Pressure Zone 3 system. Eight (8) of the 47 hydrants modeled could not meet the minimum flow requirements for each of the associated land uses. This represents approximately 17 percent of the hydrants within the pressure zone. Approximately 15 percent of the modeled hydrants in Pressure Zone 3 were unable to provide a minimum fire flow of 1,500 gpm at the required residual pressure of 20 psi. However, when the 1,500 gpm fire flow demand was split between two hydrants, the required residual pressure of 20 psi could be met, with the exception of two locations, shown on Figure 7-20.

With the previously recommended addition of two 1,750 gpm fire flow pumps to the Zone 3 pump station, the fire flow simulation results indicate that the existing system in Pressure Zone 3 is able to meet the required residual pressure of 20 psi under maximum day plus fire flow demand conditions, see Figure 7-21.

The required upgrades to the existing system for Zone 3 are extensive, and completion of pipeline upgrades for the sole purpose of improving fire flow would be cost prohibitive to the existing customers of the City. Therefore, it is not recommended that all the improvements necessary to increase fire flow capacity in the existing water system be identified as projects in the City's CIP. Instead, the improvement projects have been prioritized based on the size and age of the existing pipelines. The highest priority projects are included in the CIP and consist of correcting existing fire flow deficiencies by replacing existing 4-inch diameter water mains constructed prior to 1960 (2,975 lineal feet) followed by 6-inch diameter water mains constructed prior to 1960 (5,329 lineal feet) with 8-inch diameter pipelines. Figure 7-22 presents the available fire flow at 20 psi residual pressure with the new pumps and pipeline upgrades.

Figure 7-8 illustrates the location of 4-inch and 6-inch diameter pipelines that were constructed prior to 1960.

7.4.2.4 Pressure Zone 4

7.4.2.4.1 Peak Hour

During a peak hour demand condition, results indicate that the existing system in Pressure Zone 4 is adequate to deliver peak hour demands under the City's minimum pressure criteria of 40 psi (see Figure 7-23). System pressures in Pressure Zone 4 range from 60 to 97 psi.

Head losses in Pressure Zone 4 meet the City's maximum design criteria of 10 ft/kft.

7.4.2.4.2 Maximum Day plus Fire Flow Demand

Before running maximum day plus fire flow demand analysis, all 4-inch diameter pipelines in Zone 4 were upsized to 8-inch diameter pipelines. There were only two pipelines comprising 12 linear feet, so no pipelines were upsized.

Fire flows were simulated at hydrant locations throughout Zone 4 based on land use type. Figure 7-24 represents the residual pressures at Zone 4 hydrants during maximum day plus fire flow conditions. Results indicate that the existing system in Pressure Zone 4 is able to meet the required residual pressure of 20 psi under maximum day plus fire flow demand.

Figure 7-8 illustrates the location of 6-inch diameter pipelines that were constructed prior to 1960. The total length of the 6-inch diameter pipeline prior to 1960 is 361 lineal feet.

7.5 SUMMARY OF RECOMMENDED IMPROVEMENTS FOR EXISTING POTABLE WATER SYSTEM

The recommended improvements needed to eliminate deficiencies identified in the analysis of the existing distribution systems are summarized below.

7.5.1 Water Storage

Install a remote controlled shut-off valve or seismic valve on each of the three reservoirs to prevent drainage after a significant earthquake.

7.5.2 Pump Stations

The addition of two 1,750 gpm fire flow pumps to the Zone 3 pump station, to resolve the City's firm pumping capacity deficiency, and to assist Zone 3 in meeting its fire flow requirements.

7.5.3 Pipelines

The following improvements are recommended:

<u>Peak Hour Improvements</u>

- Reconfigure a portion of the Southwest corner of Pressure Zone 1 so that it is served by Pressure Zone 2. Construct approximately 450 feet of 8-inch diameter pipeline from existing 8-inch diameter pipeline along Kellogg Lake Apartments to SE River Road. Isolate 6-inch diameter pipeline along SE 22nd Avenue from Zone 1. The exact locations to isolate this pipeline should be verified by City field staff.

<u>Fire Flow Improvements in Areas Zoned "Public"</u>

- Upsize approximately 320 feet of existing 6-inch diameter pipeline to 8-inch diameter pipeline from the hydrant to Willard Street in Zone 1.
- Upsize approximately 600 feet of existing 6-inch diameter pipeline to 8-inch diameter pipeline and upsize approximately 95 feet of existing 4-inch diameter pipeline to 8-inch diameter pipeline in the area to the west of Flavel Drive in Zone 2

<u>Fire Flow Improvements to 4" Pipelines Constructed Prior to 1960</u>

- Replace approximately 10 lineal feet of 4-inch diameter pipeline in pressure Zone 1 with 8-inch diameter pipeline, see Figure 7-8.
- Replace approximately 10,582 lineal feet of 4-inch diameter pipeline in pressure Zone 2 with 8-inch diameter pipeline, see Figure 7-8.
- Replace approximately 2,975 lineal feet of 4-inch diameter pipeline in pressure Zone 3 with 8-inch diameter pipeline, see Figure 7-8.

Fire Flow Improvements to 6” Pipelines Constructed Prior to 1960

- Replace approximately 15,156 lineal feet of 6-inch diameter pipeline in Pressure Zone 1 with 8-inch diameter pipeline, see Figure 7-8.
- Replace approximately 49,373 lineal feet of 6-inch diameter pipeline in Pressure Zone 2 with 8-inch diameter pipeline, see Figure 7-8.
- Replace approximately 5,329 lineal feet of 6-inch diameter pipeline in Pressure Zone 3 with 8-inch diameter pipeline, see Figure 7-8.
- Replace approximately 361 lineal feet of 6-inch diameter pipeline in Pressure Zone 4 with 8-inch diameter pipeline, see Figure 7-8.

General Fire Flow Improvements

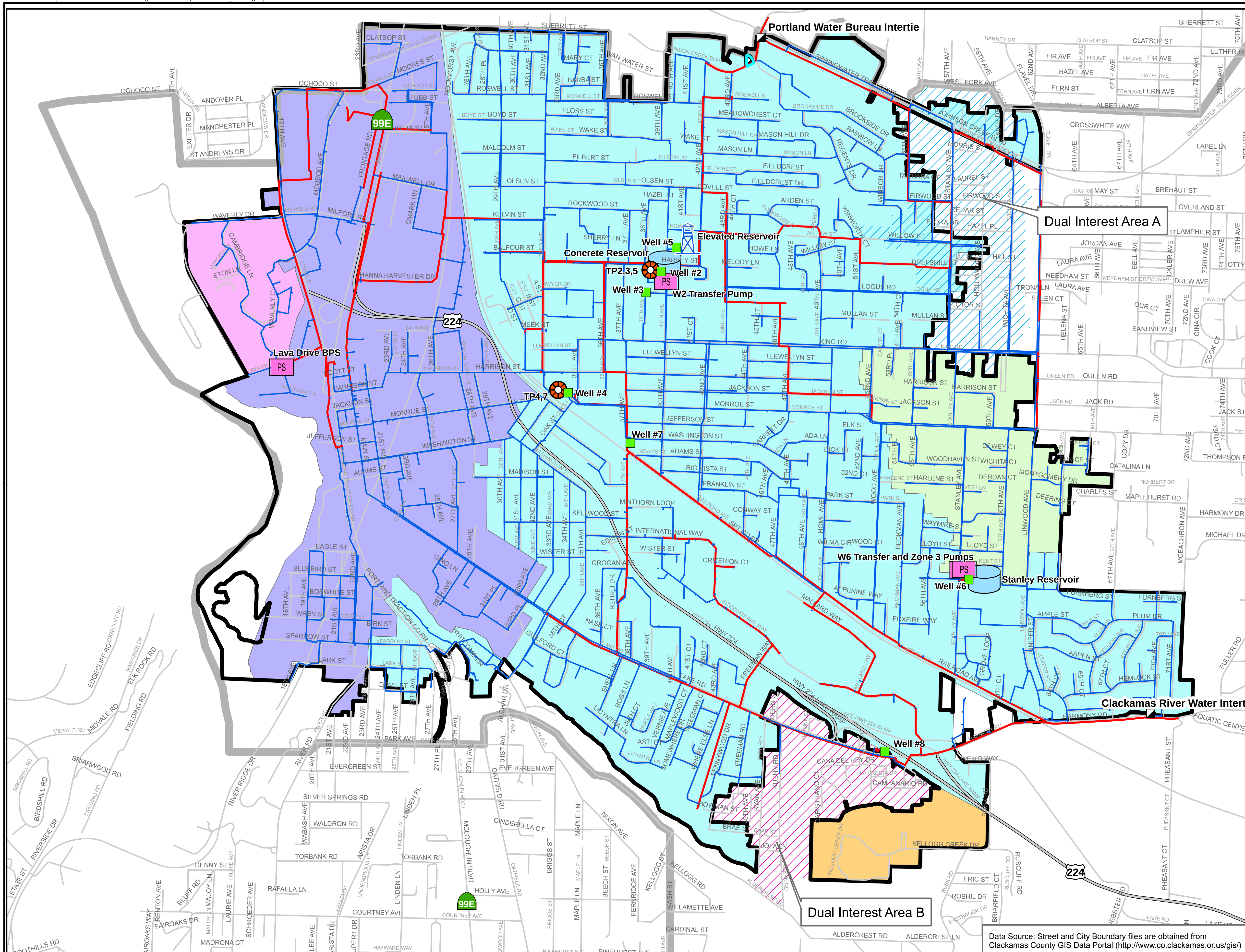
- Improve fire flow capacity in the existing water system as part of future pipeline replacement projects.

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-1_Existing System.mxd 8/22/2011

FIGURE 7-1

City of Milwaukie
2010 Water System Master Plan

2010 WATER SYSTEM



LEGEND

- Pump Station
- At Grade Reservoir
- Elevated Reservoir
- Treatment Tower
- Well
- Emergency Interties
- Pipeline Diameter < 12 inches
- Pipeline Diameter ≥ 12 inches
- City Boundary
- Urban Growth Boundary
- Dual Interest Area A
- Dual Interest Area B

City's Pressure Zone Boundary

- Zone 1
- Zone 2
- Zone 3
- Zone 4
- Clackamas River Water
- Street

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O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-2_Land Use.mxd 4/29/2011

FIGURE 7-2

City of Milwaukie
2010 Water System Master Plan

2010 GENERAL PLAN LAND USE

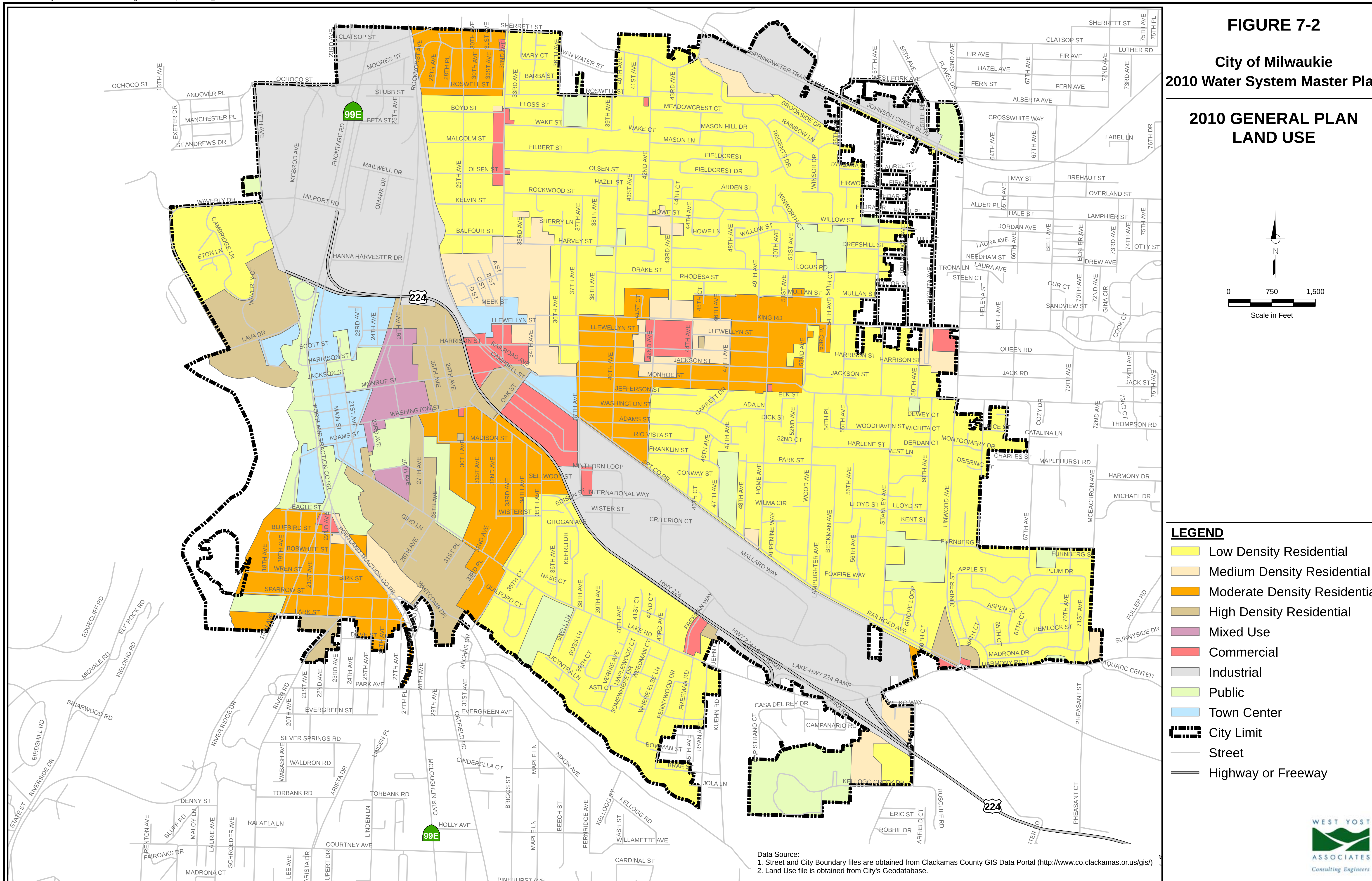
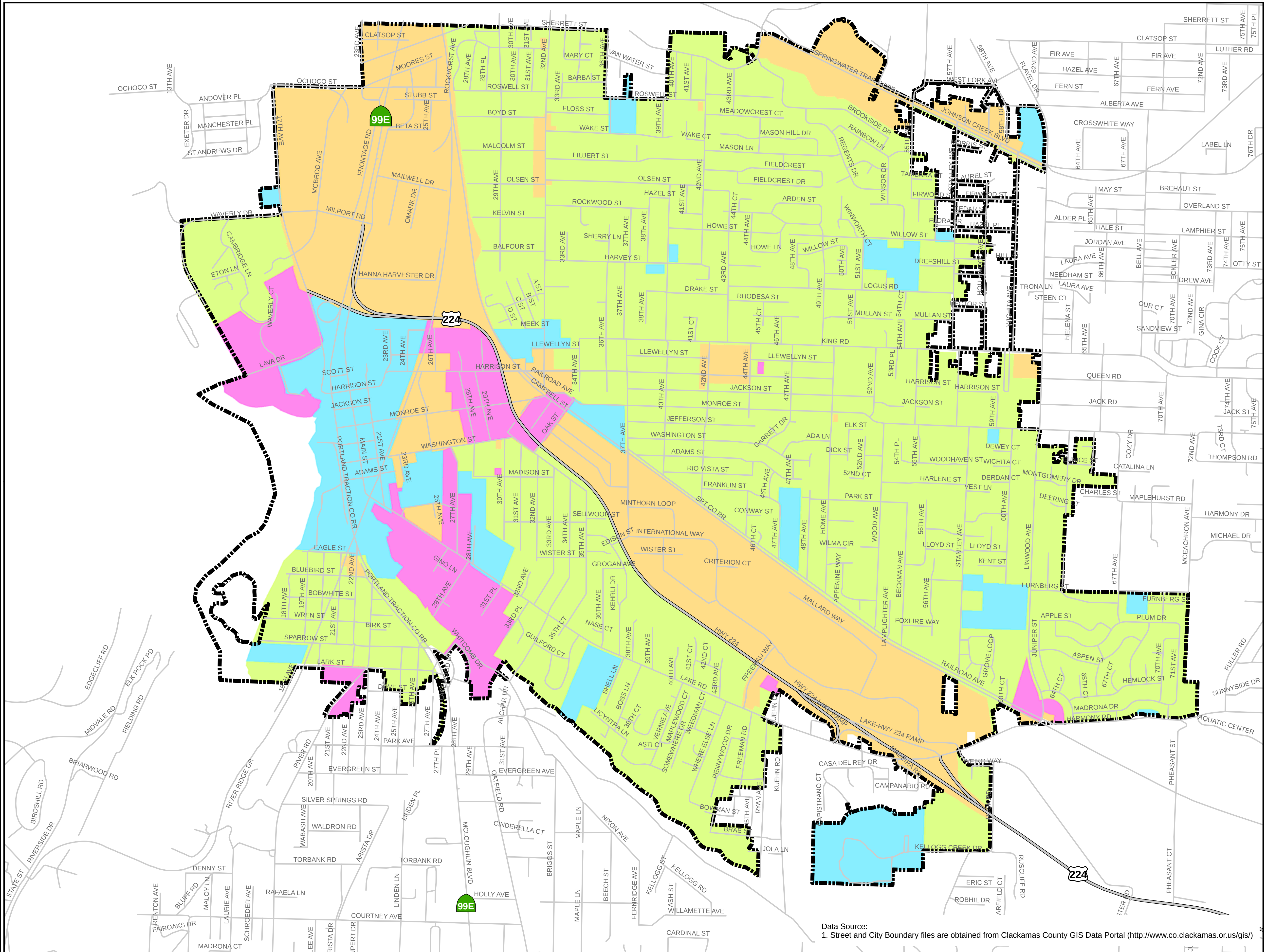


FIGURE 7-3

City of Milwaukie
2010 Water System Master Plan

2010 WATER SYSTEM FIRE FLOW DEMANDS



Scale in Feet: 0 — 750 — 1,500

LEGEND

Fire Flow Demand

- 1,500 gpm
- 1,500 gpm
- 2,000 gpm
- 3,000 gpm
- City Limit
- Street
- Highway or Freeway

Data Source:
1. Street and City Boundary files are obtained from Clackamas County GIS Data Portal (http://www.co.clackamas.or.us/gis/)

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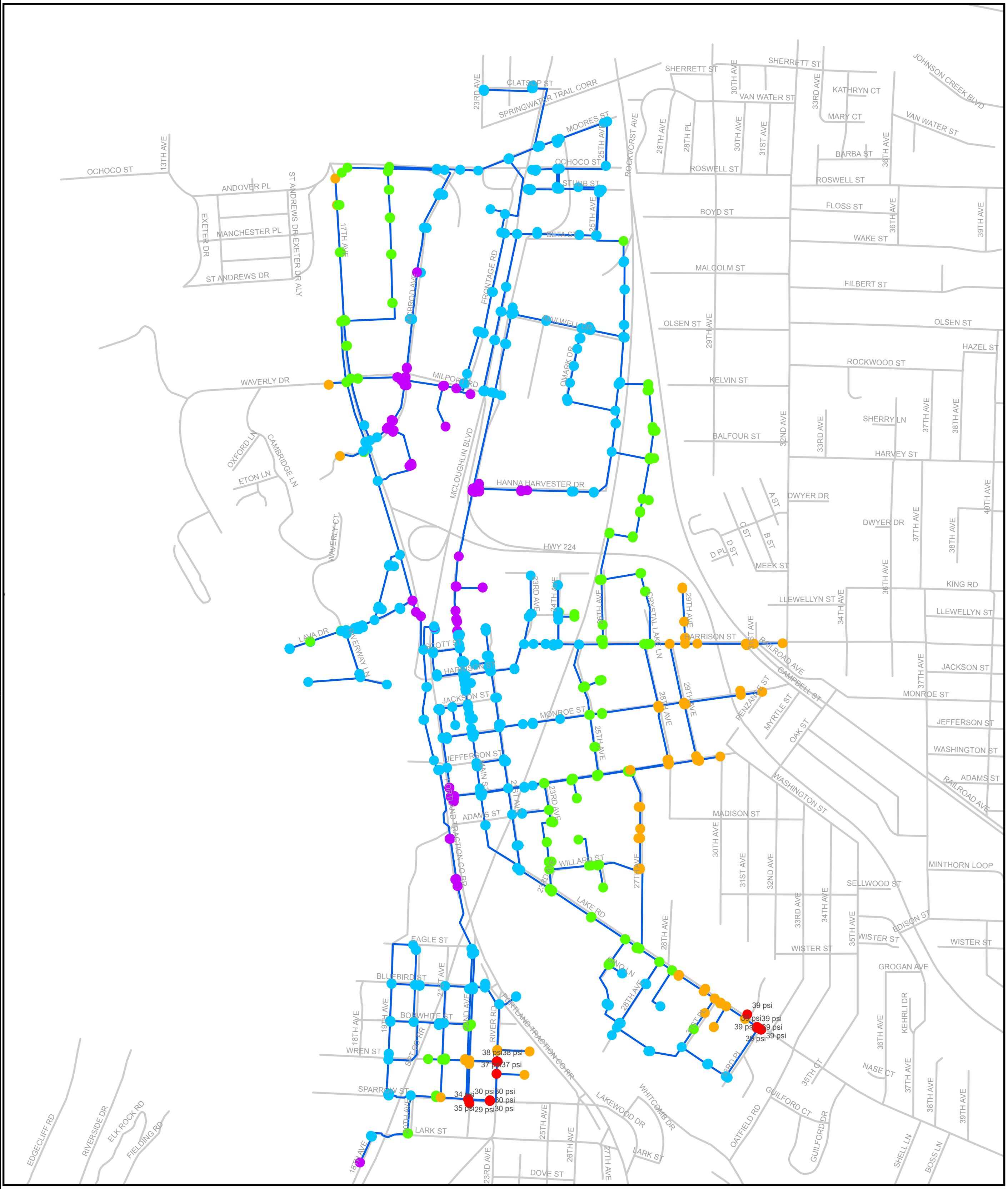
O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-4_PH_Zone1.mxd



LEGEND

Red dot	Pressure < 40 psi
Orange dot	40 psi ≤ Pressure < 50 psi
Green dot	50 psi ≤ Pressure < 60 psi
Blue dot	60 psi ≤ Pressure < 70 psi
Purple dot	Pressure ≥ 70 psi
Blue line	HL/1000 < 10 ft/kft
Red line	HL/1000 ≥ 10 ft/kft
Grey line	Street

N

0 650 1,300

Scale in Feet

FIGURE 7-4

City of Milwaukie
2010 Water System Master Plan
PRESSURE ZONE 1
PEAK HOUR ANALYSIS

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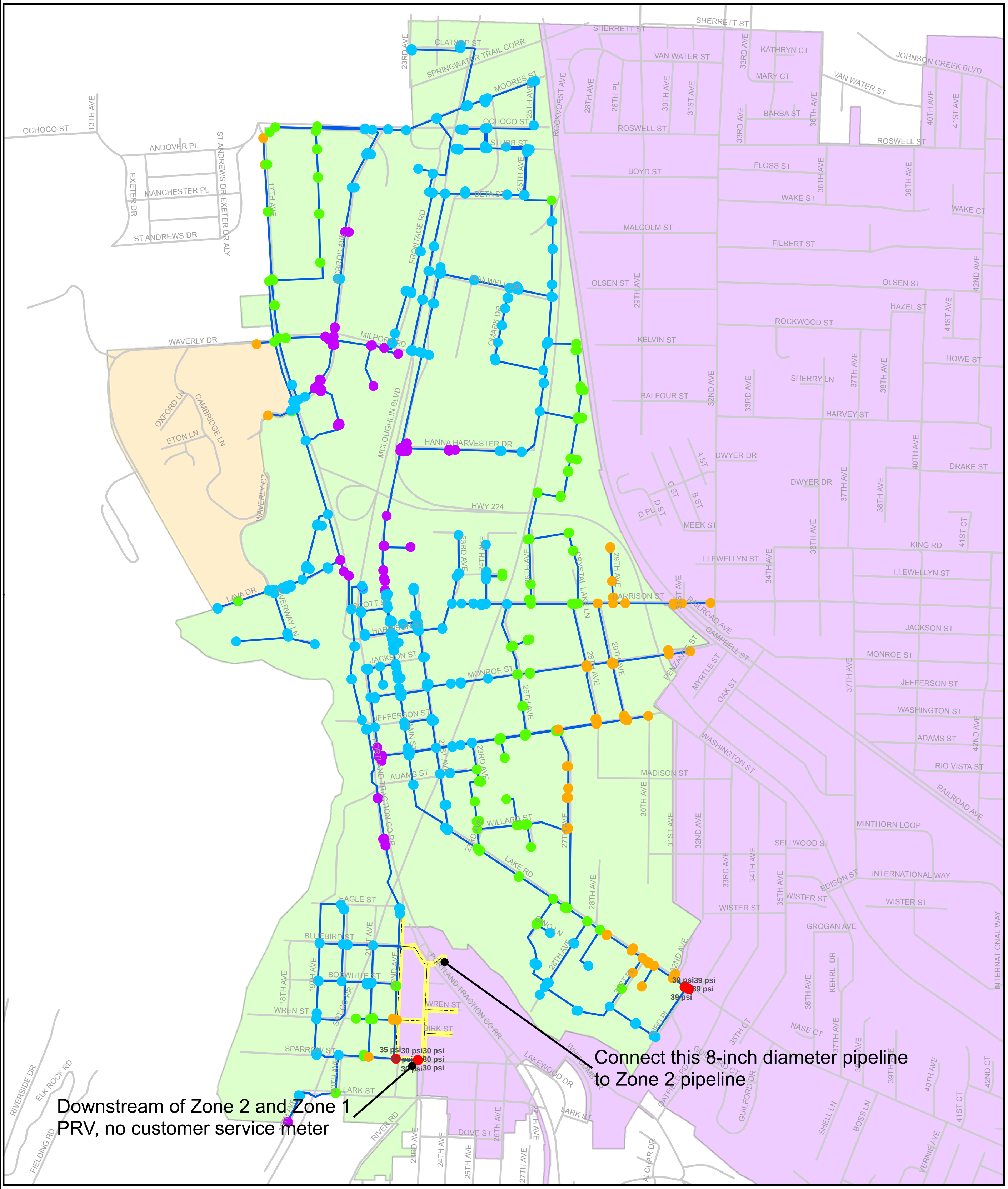
O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-5_PH_Zone1+Rezone.mxd



LEGEND

- Pressure < 40 psi
- 40 psi ≤ Pressure < 50 psi
- 50 psi ≤ Pressure < 60 psi
- 60 psi ≤ Pressure < 70 psi
- Pressure ≥ 70 psi
- HL/1000 < 10 ft/kft
- HL/1000 ≥ 10 ft/kft
- Pipeline Moved to Zone 2
- Street

N

0 650 1,300

Scale in Feet

FIGURE 7-5

City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 1
PEAK HOUR ANALYSIS FOR
REZONE OF ZONE 1 AT
RIVER ROAD AND SPARROW STREET

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O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-6_MDFF_Z1.mxd

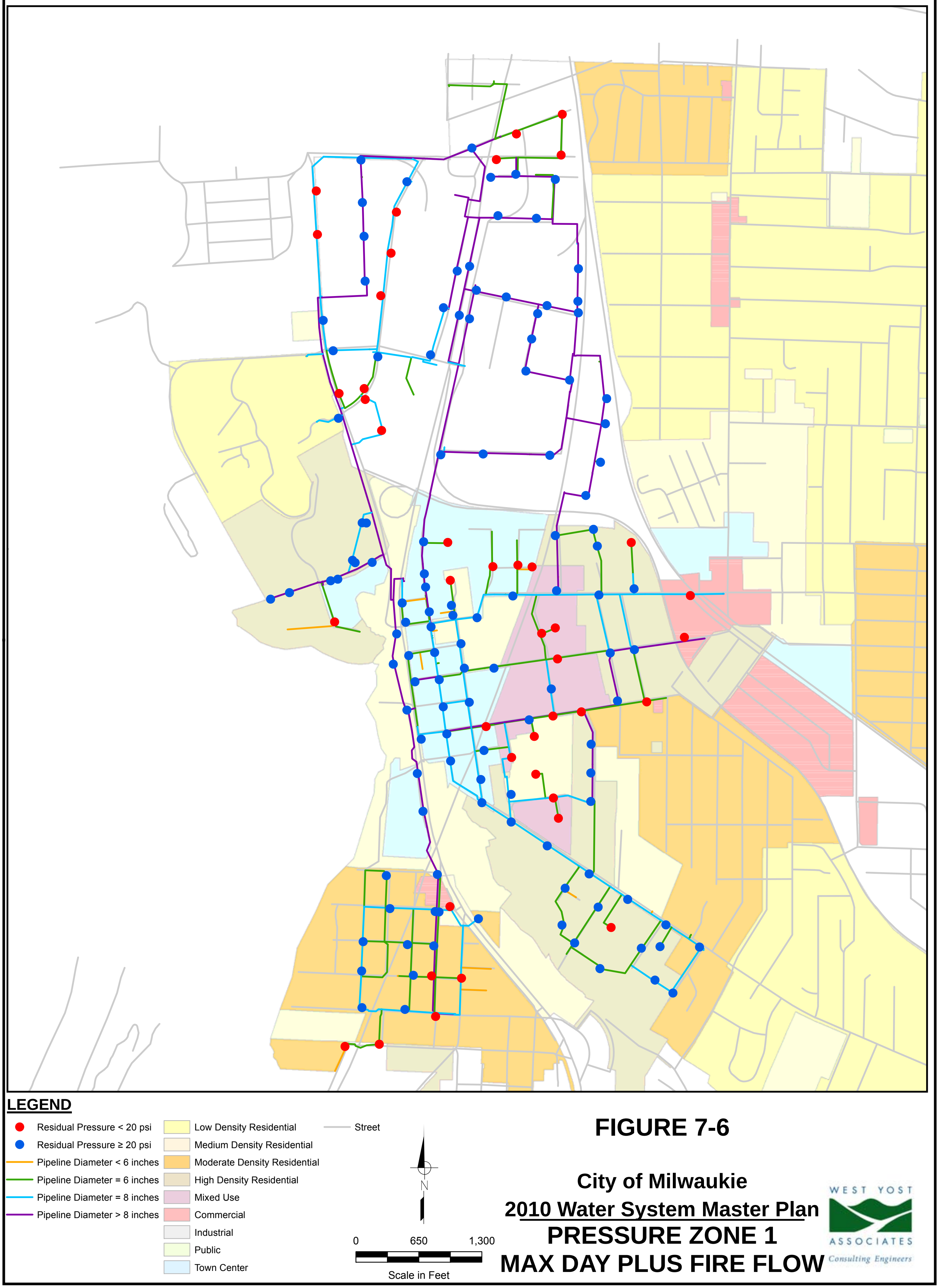


FIGURE 7-6

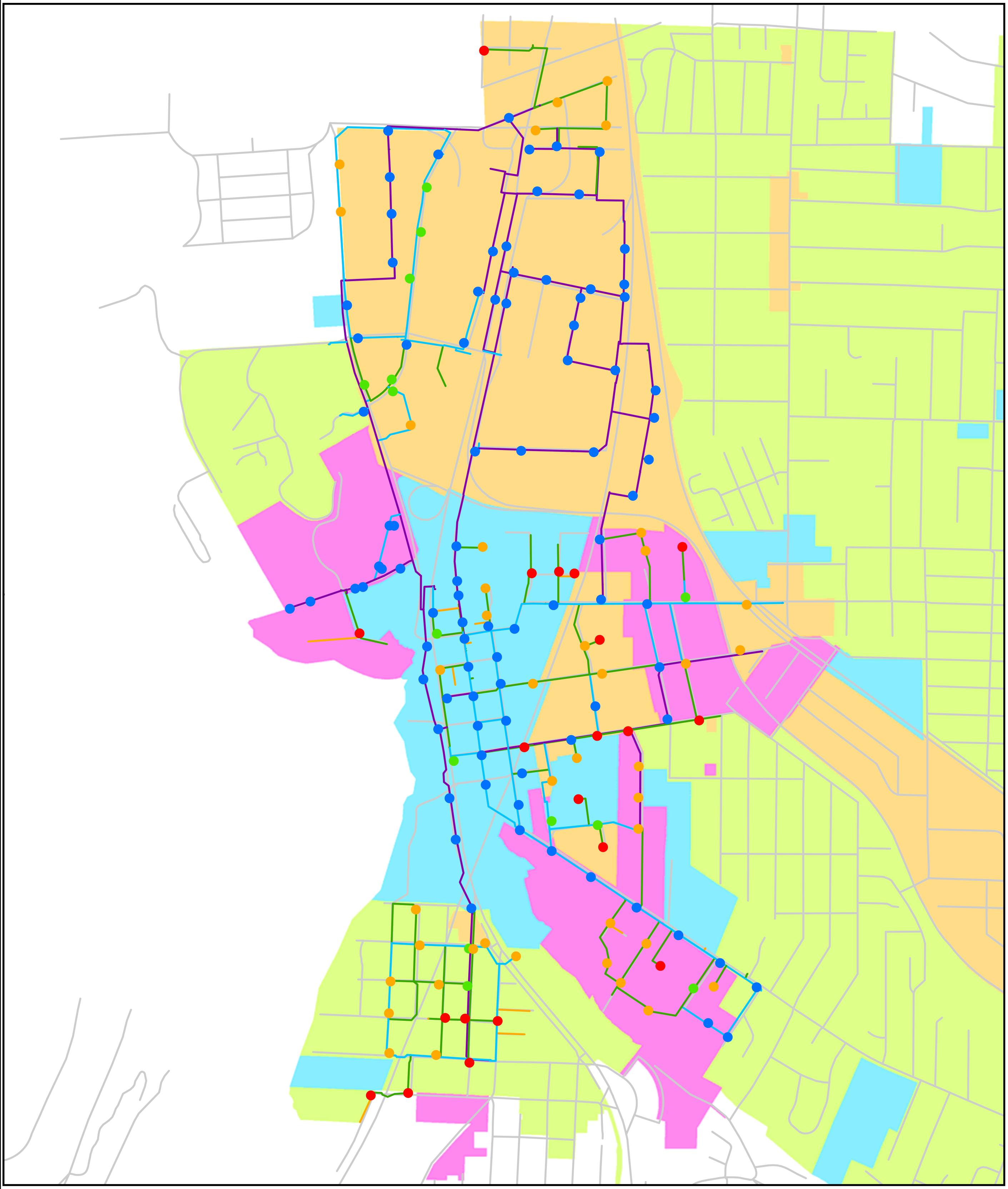
City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 1

MAX DAY PLUS FIRE FLOW

WEST YOST ASSOCIATES Consulting Engineers



LEGEND

- Fire Flow < 1,500 gpm
- 1,500 gpm ≤ Fire Flow < 2,500 gpm
- 2,500 gpm ≤ Fire Flow < 3,000 gpm
- Fire Flow ≥ 3,000 gpm
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches

Fire Flow Demand

- 1,500 gpm for 2 hours
- 1,500 gpm for 3 hours
- 2,000 gpm for 4 hours
- 3,000 gpm for 4 hours

- Street

N

0 650 1,300

Scale in Feet

FIGURE 7-7

City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 1

AVAILABLE FIRE FLOW

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O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-8_Pipe_Age.mxd

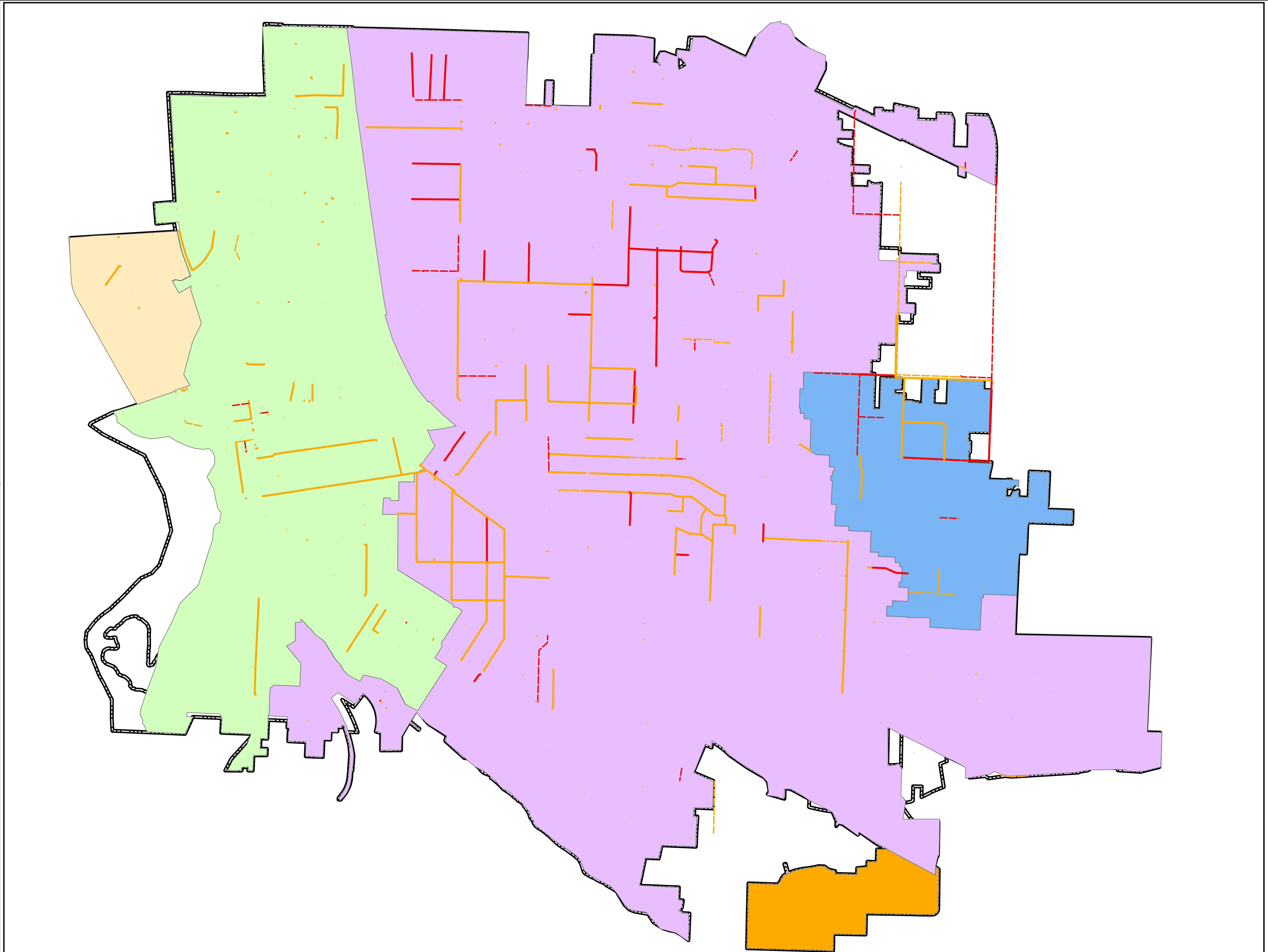

FIGURE 7-8

City of Milwaukie
2010 Water System Master Plan

EXISTING WATER SYSTEM RECOMMENDED RENEWAL AND REPLACEMENT PROGRAM

N

0 700 1,400

Scale in Feet

LEGEND

Pipeline Diameter-Year Installed

- 4-inch Diameter, Pre-1960
- 6-inch Diameter, Pre-1960
- 4-inch Diameter, Year Unknown
- 6-inch Diameter, Year Unknown

City's Pressure Zone Boundary

- Zone 1
- Zone 2
- Zone 3
- Zone 4
- Served by CRW
- City Boundary

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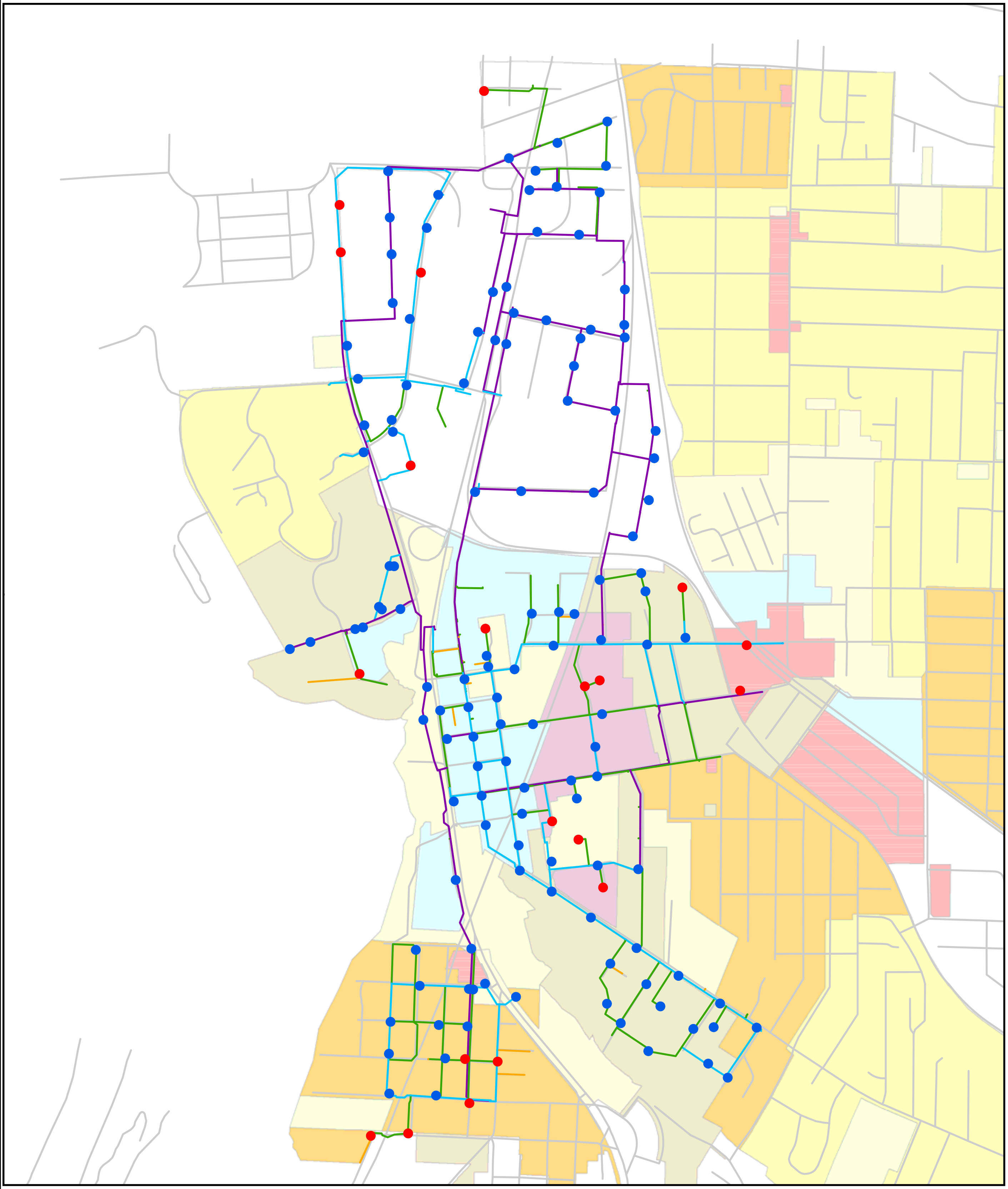
O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-9_MDFF_Z1+FIX(4&6_1960).mxd



LEGEND

- Residual Pressure < 20 psi
- Residual Pressure ≥ 20 psi
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches
- Low Density Residential
- Medium Density Residential
- Moderate Density Residential
- High Density Residential
- Mixed Use
- Commercial
- Industrial
- Public
- Town Center
- Street

N

0 650 1,300

Scale in Feet

FIGURE 7-9

City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 1

MAX DAY PLUS FIRE FLOW

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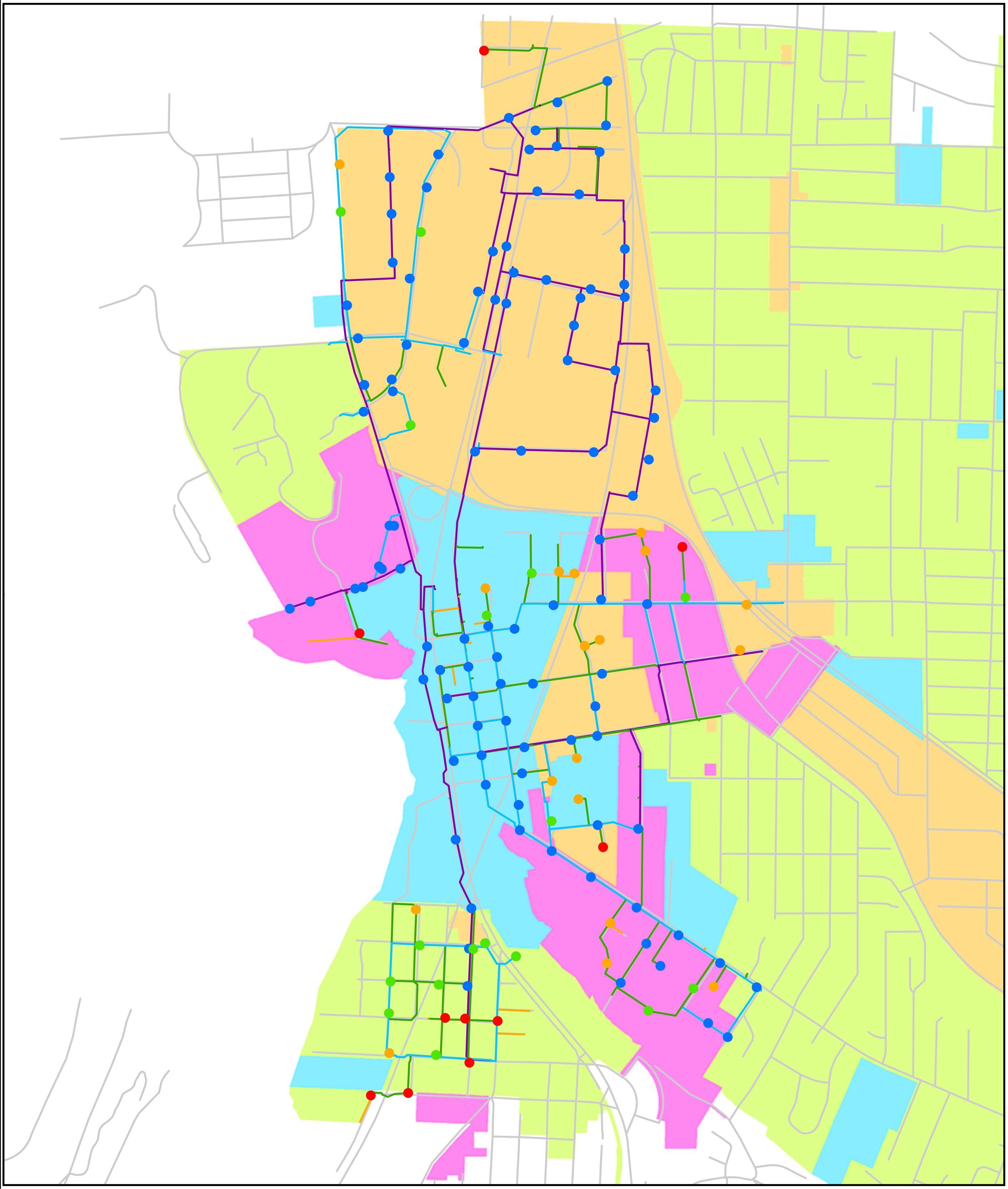
O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-10_AvailFF_Z1.mxd



LEGEND

- Fire Flow < 1,500 gpm
- 1,500 gpm ≤ Fire Flow < 2,500 gpm
- 2,500 gpm ≤ Fire Flow < 3,000 gpm
- Fire Flow ≥ 3,000 gpm
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches

Fire Flow Demand

- 1,500 gpm for 2 hours
- 1,500 gpm for 3 hours
- 2,000 gpm for 4 hours
- 3,000 gpm for 4 hours

- Street

N

0 650 1,300

Scale in Feet

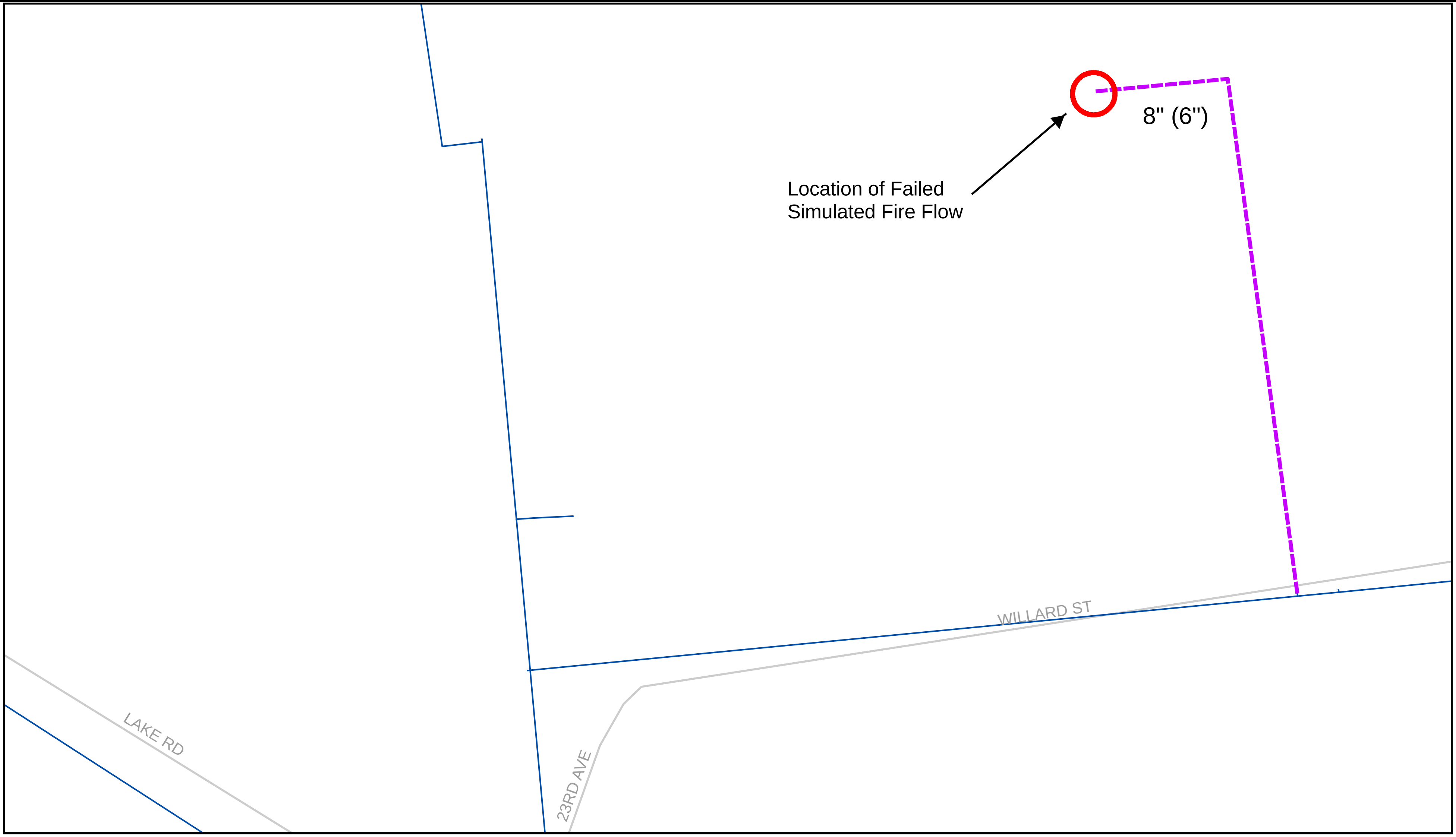
FIGURE 7-10

City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 1
AVAILABLE FIRE FLOW

WEST YOST ASSOCIATES
Consulting Engineers



LEGEND

Hydrant

Existing Pipeline

Increased Pipeline Diameter

Street

8" (6") Recommended (Existing) Diameter

N

0 35 70

Scale in Feet

FIGURE 7-11

City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 1

FIRE FLOW RECOMMENDATION #1

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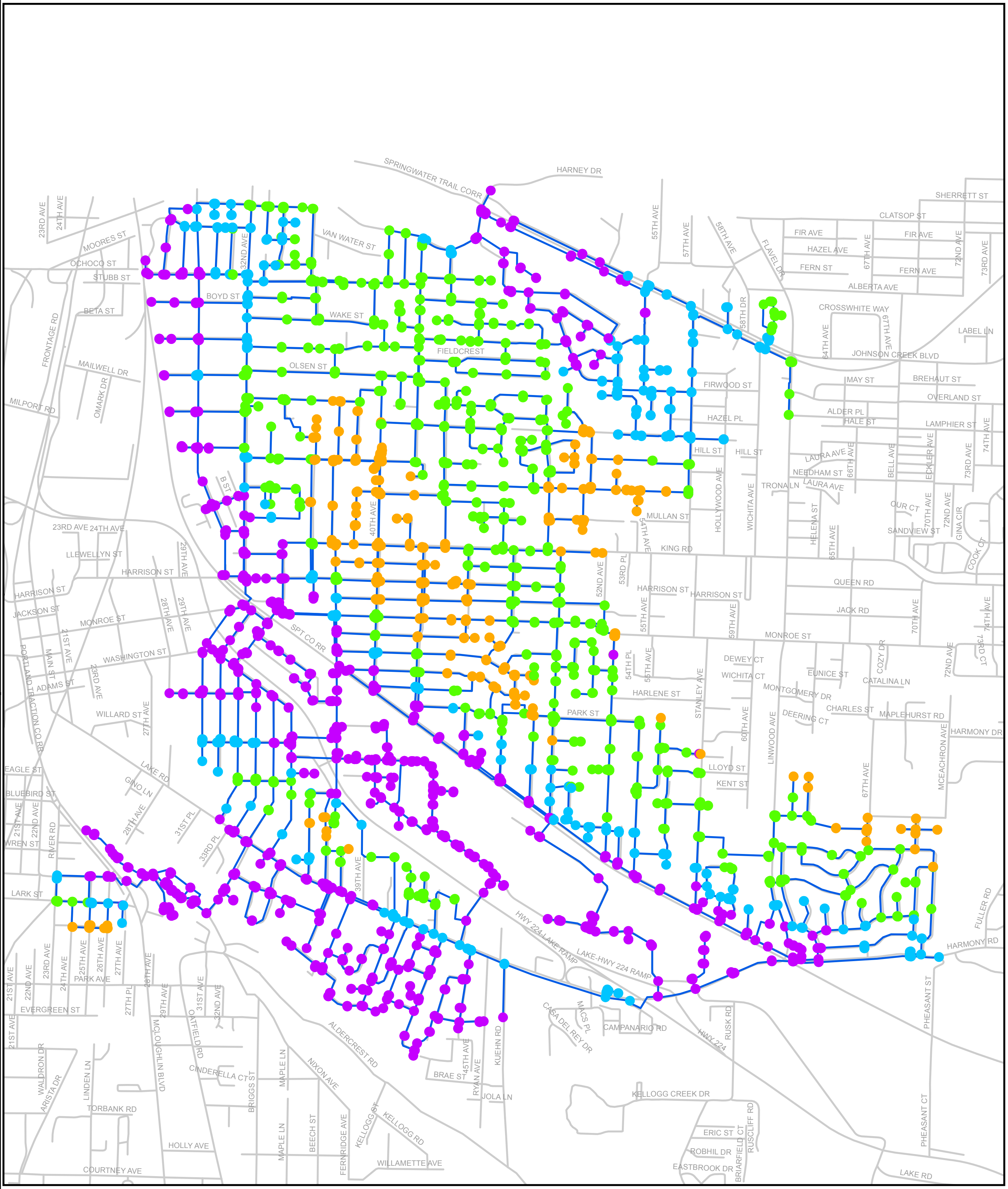
O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-11_Z1_FFRec_1.mxd 8/22/2011

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-12_PH_Z2.mxd 8/22/2011



LEGEND

- Pressure < 40 psi
- 40 psi ≤ Pressure < 50 psi
- 50 psi ≤ Pressure < 60 psi
- 60 psi ≤ Pressure < 70 psi
- Pressure ≥ 70 psi
- HL/1000 < 10 ft/kft
- HL/1000 ≥ 10 ft/kft
- Street

N

0 1,000 2,000

Scale in Feet

FIGURE 7-12

City of Milwaukie

2010 Water System Master Plan

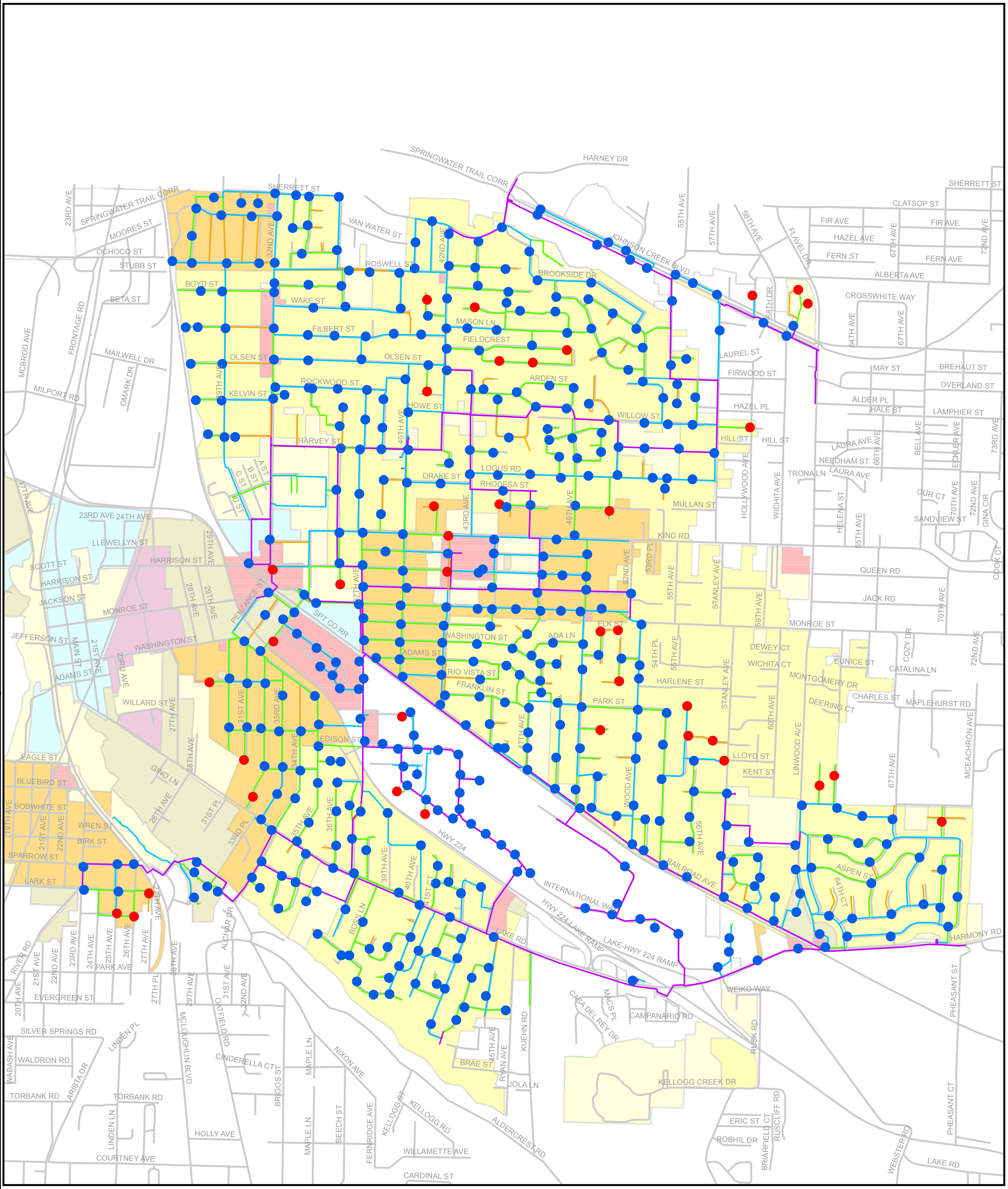
PRESSURE ZONE 2

PEAK HOUR ANALYSIS

WEST YOST ASSOCIATES

Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-13_MDFF_Z2.mxd 8/22/2011



LEGEND

- Residual Pressure < 20 psi
- Residual Pressure ≥ 20 psi
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches
- Low Density Residential
- Medium Density Residential
- Moderate Density Residential
- High Density Residential
- Mixed Use
- Commercial
- Industrial
- Public
- Town Center
- Street

N

0 1,000 2,000

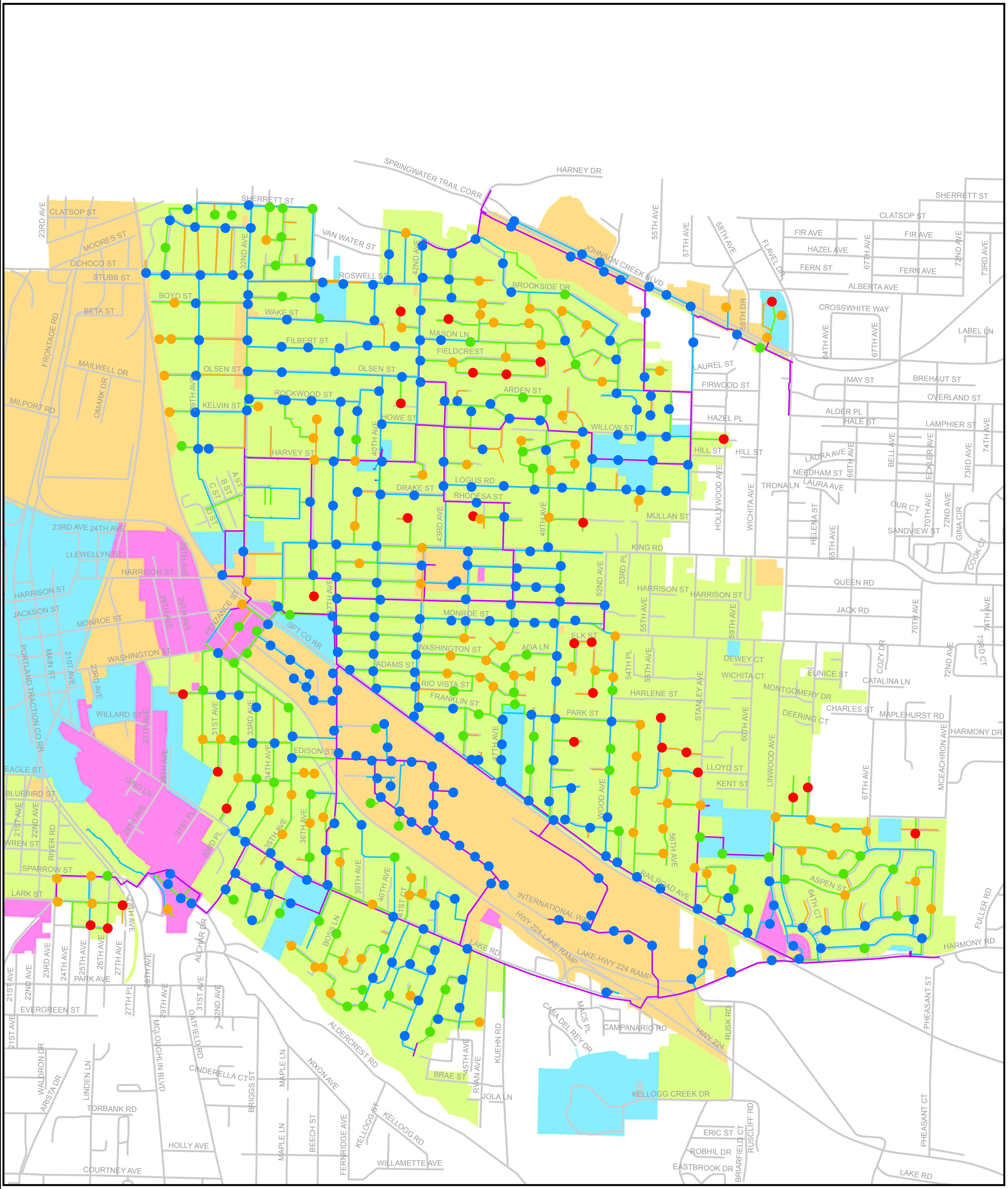
Scale in Feet

FIGURE 7-13

City of Milwaukie
2010 Water System Master Plan
PRESSURE ZONE 2
MAX DAY PLUS FIRE FLOW

WEST YOST ASSOCIATES
Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-14_AvailFF_Z2.mxd 8/22/2011



LEGEND

- Fire Flow < 1,500 gpm
- 1,500 gpm ≤ Fire Flow < 2,500 gpm
- 2,500 gpm ≤ Fire Flow < 3,000 gpm
- Fire Flow ≥ 3,000 gpm
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches

Fire Flow Demand

- 1,500 gpm for 2 hours
- 1,500 gpm for 3 hours
- 2,000 gpm for 4 hours
- 3,000 gpm for 4 hours

- Street

N

0 1,000 2,000

Scale in Feet

FIGURE 7-14

City of Milwaukie

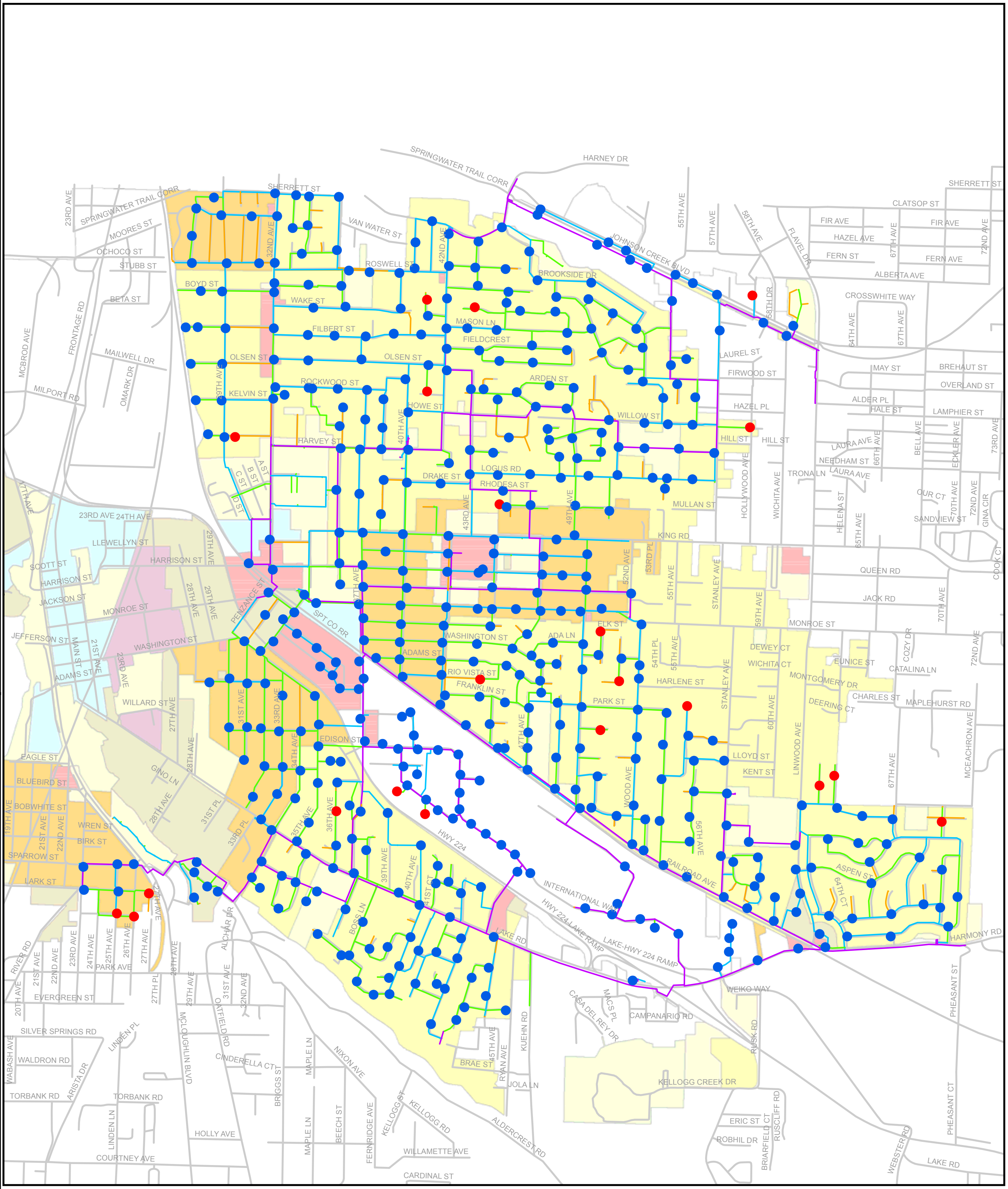
2010 Water System Master Plan

PRESSURE ZONE 2

AVAILABLE FIRE FLOW

WEST YOST ASSOCIATES

Consulting Engineers



LEGEND

- Residual Pressure < 20 psi
- Residual Pressure ≥ 20 psi
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches
- Low Density Residential
- Medium Density Residential
- Moderate Density Residential
- High Density Residential
- Mixed Use
- Commercial
- Industrial
- Public
- Town Center
- Street

N

0 1,000 2,000

Scale in Feet

FIGURE 7-15

City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 2

MAX DAY PLUS FIRE FLOW

WEST YOST ASSOCIATES

Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-16_AvailFF_Z2.mxd 8/22/2011

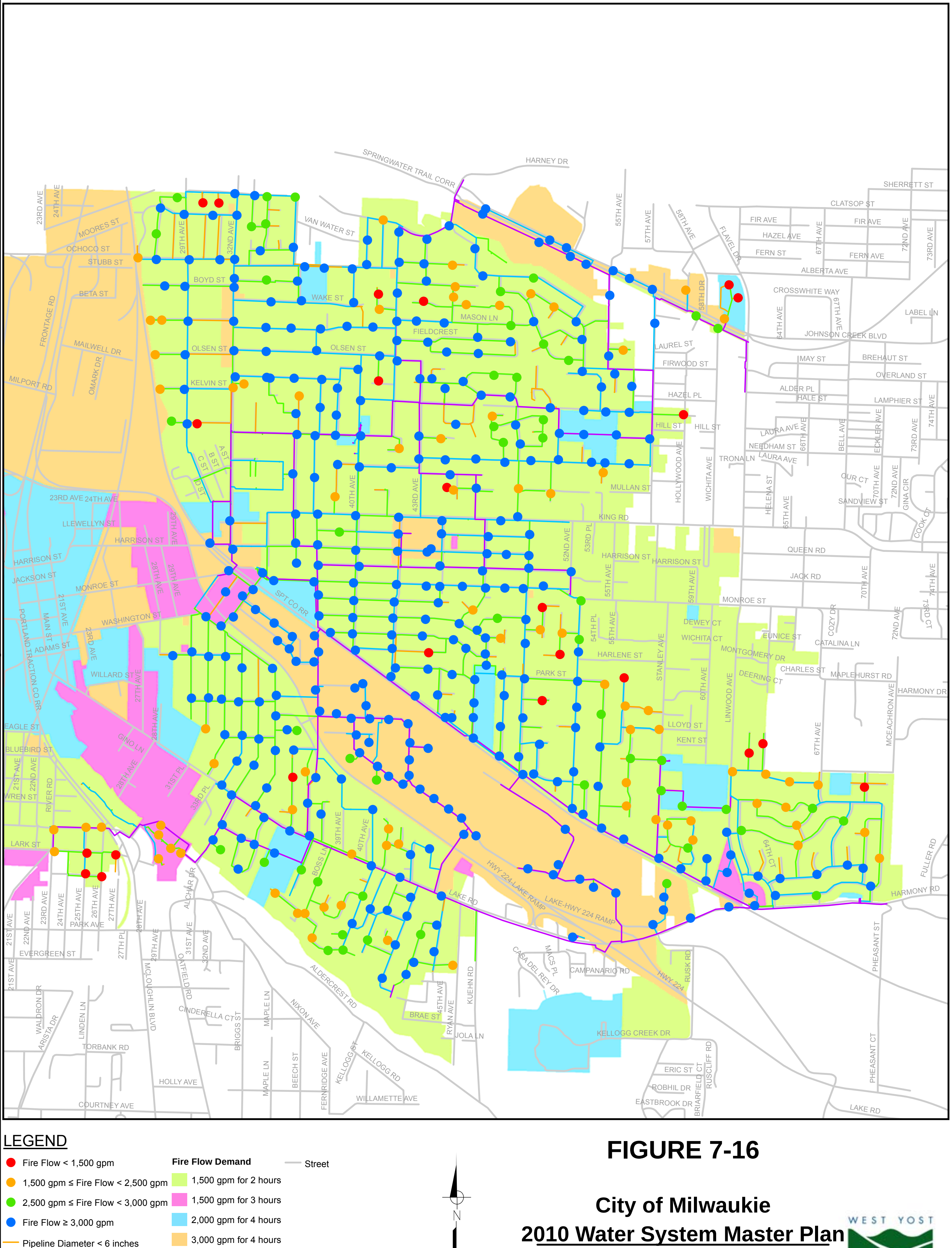


FIGURE 7-16

City of Milwaukie
2010 Water System Master Plan
PRESSURE ZONE 2
AVAILABLE FIRE FLOW

WEST YOST ASSOCIATES
Consulting Engineers

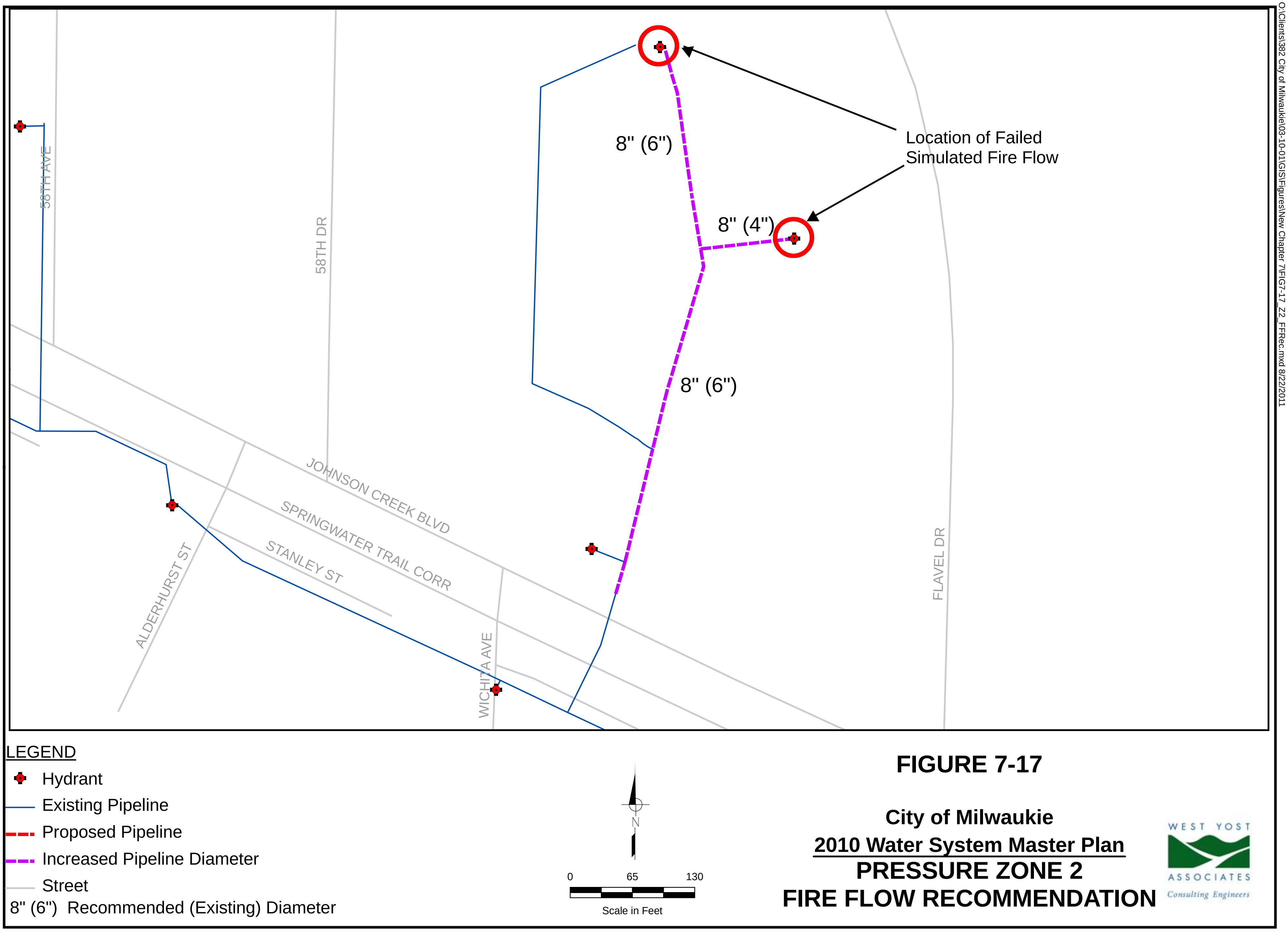


FIGURE 7-17

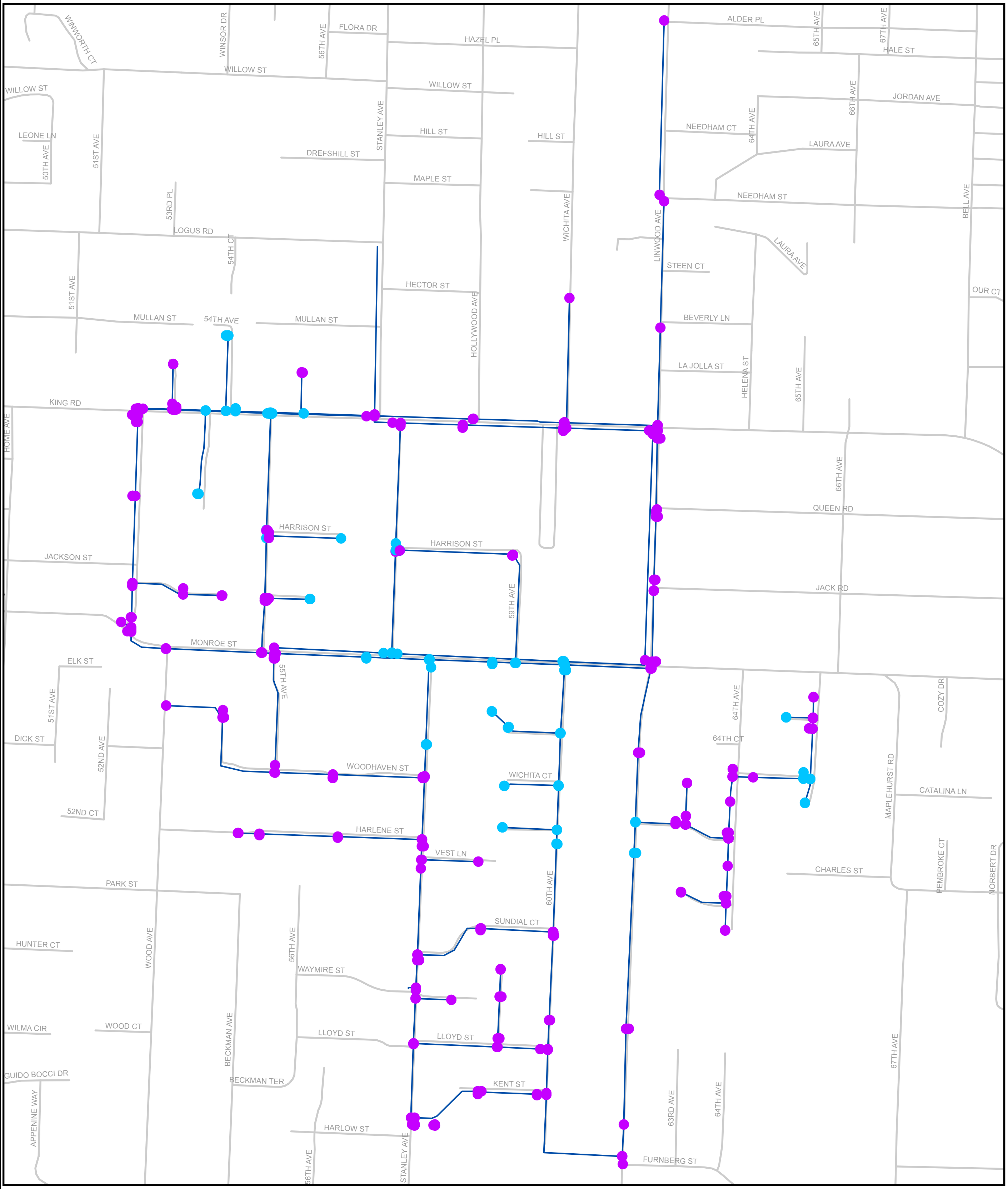
City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 2

FIRE FLOW RECOMMENDATION

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-18_PH_Z3.mxd 8/22/2011



LEGEND

- 60 psi ≤ Pressure < 70 psi
- Pressure ≥ 70 psi
- HL/1000 < 10 ft/kft
- Street

N

0 350 700

Scale in Feet

FIGURE 7-18

City of Milwaukie

2010 Water System Master Plan

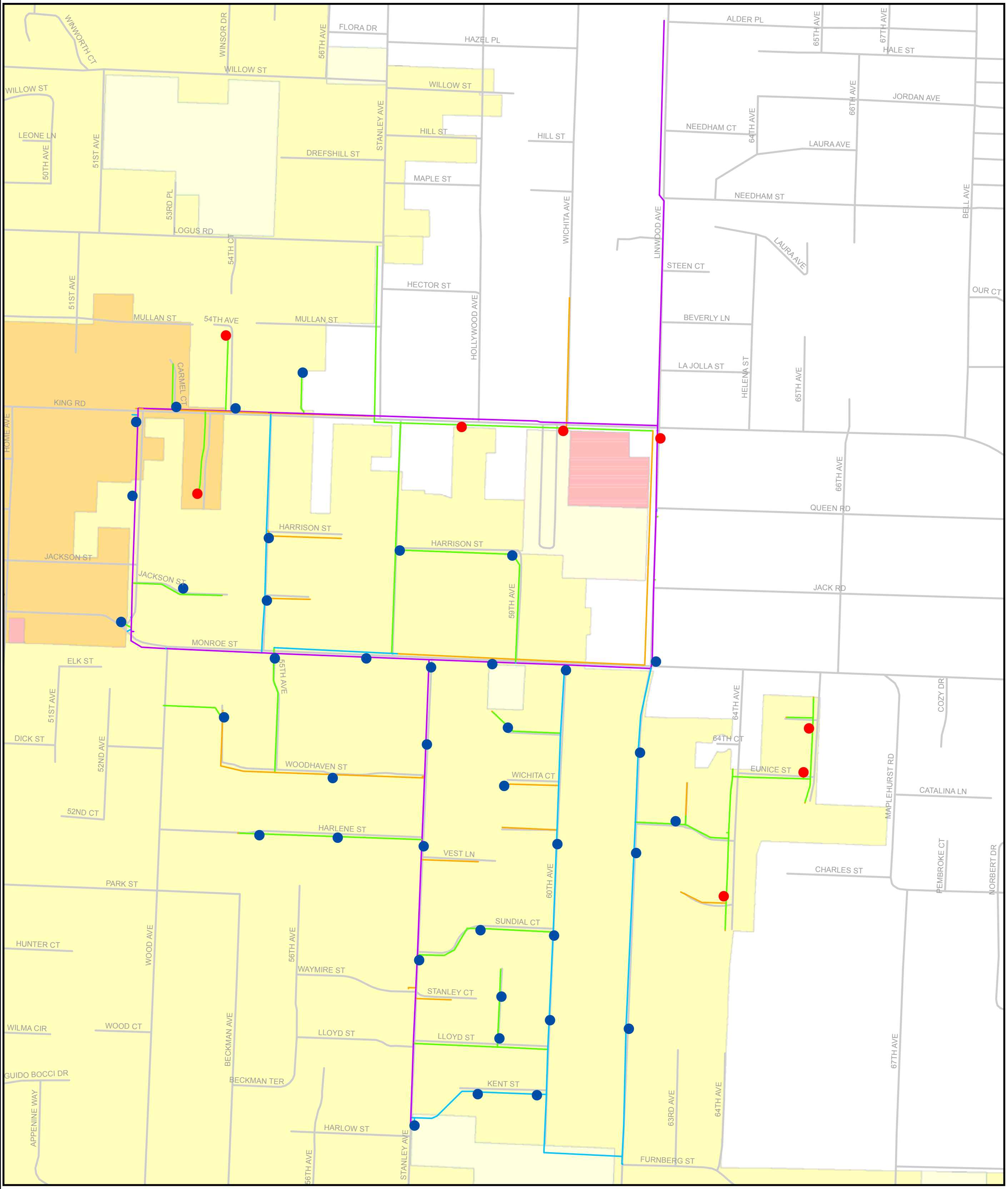
PRESSURE ZONE 3

PEAK HOUR ANALYSIS

WEST YOST ASSOCIATES

Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-19_MDFF_Z3.mxd 8/22/2011



LEGEND

- Residual Pressure < 20 psi
- Residual Pressure ≥ 20 psi
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches
- Low Density Residential
- Medium Density Residential
- Moderate Density Residential
- High Density Residential
- Mixed Use
- Commercial
- Industrial
- Public
- Town Center
- Street

N

0 350 700

Scale in Feet

FIGURE 7-19

City of Milwaukie

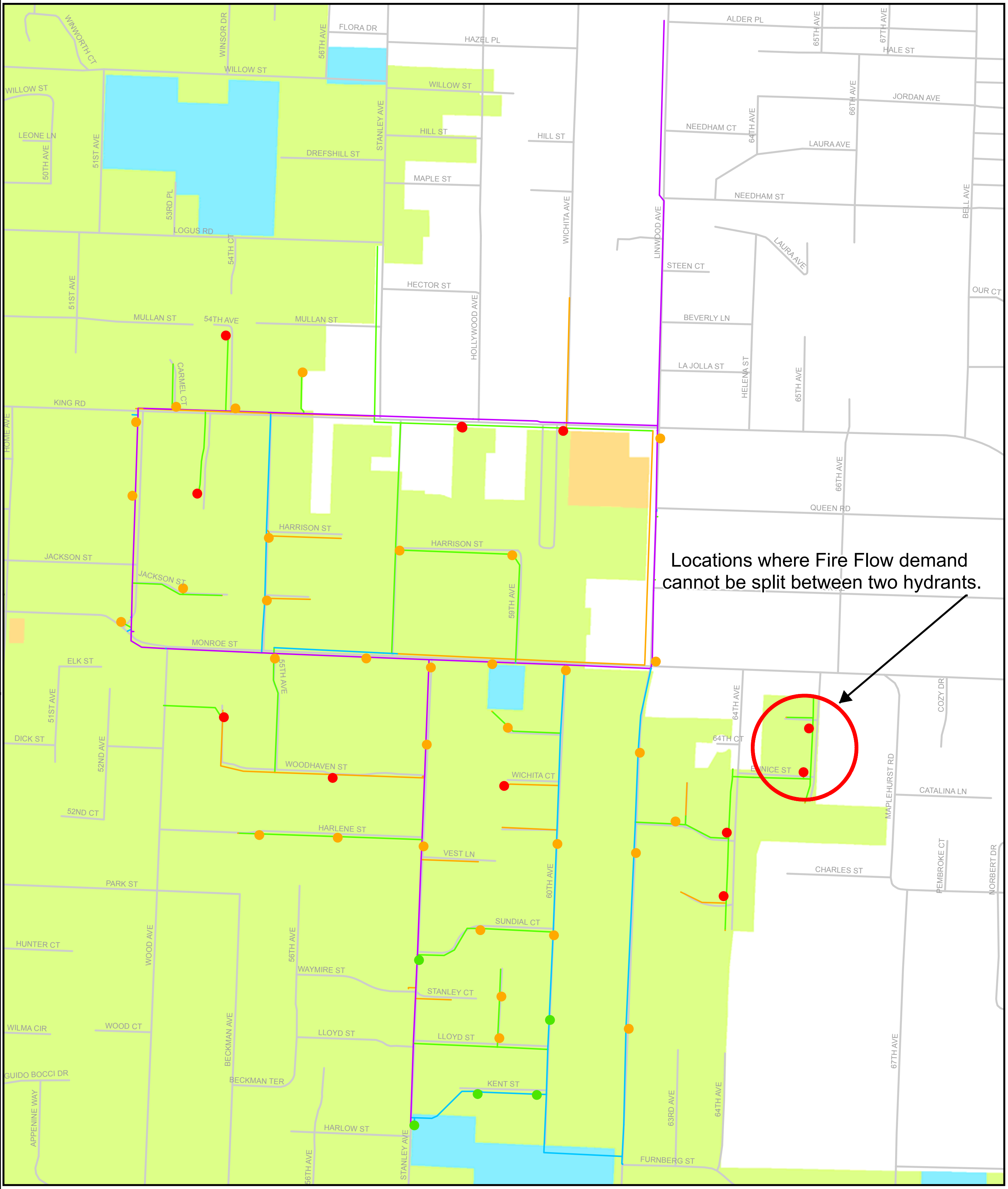
2010 Water System Master Plan

PRESSURE ZONE 3

MAX DAY PLUS FIRE FLOW

WEST YOST ASSOCIATES

Consulting Engineers



LEGEND

- Fire Flow < 1,500 gpm
- 1,500 gpm ≤ Fire Flow < 2,500 gpm
- 2,500 gpm ≤ Fire Flow < 3,000 gpm
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches

Fire Flow Demand

- 1,500 gpm for 2 hours
- 1,500 gpm for 3 hours
- 2,000 gpm for 4 hours
- 3,000 gpm for 4 hours

- Street

N

0 350 700

Scale in Feet

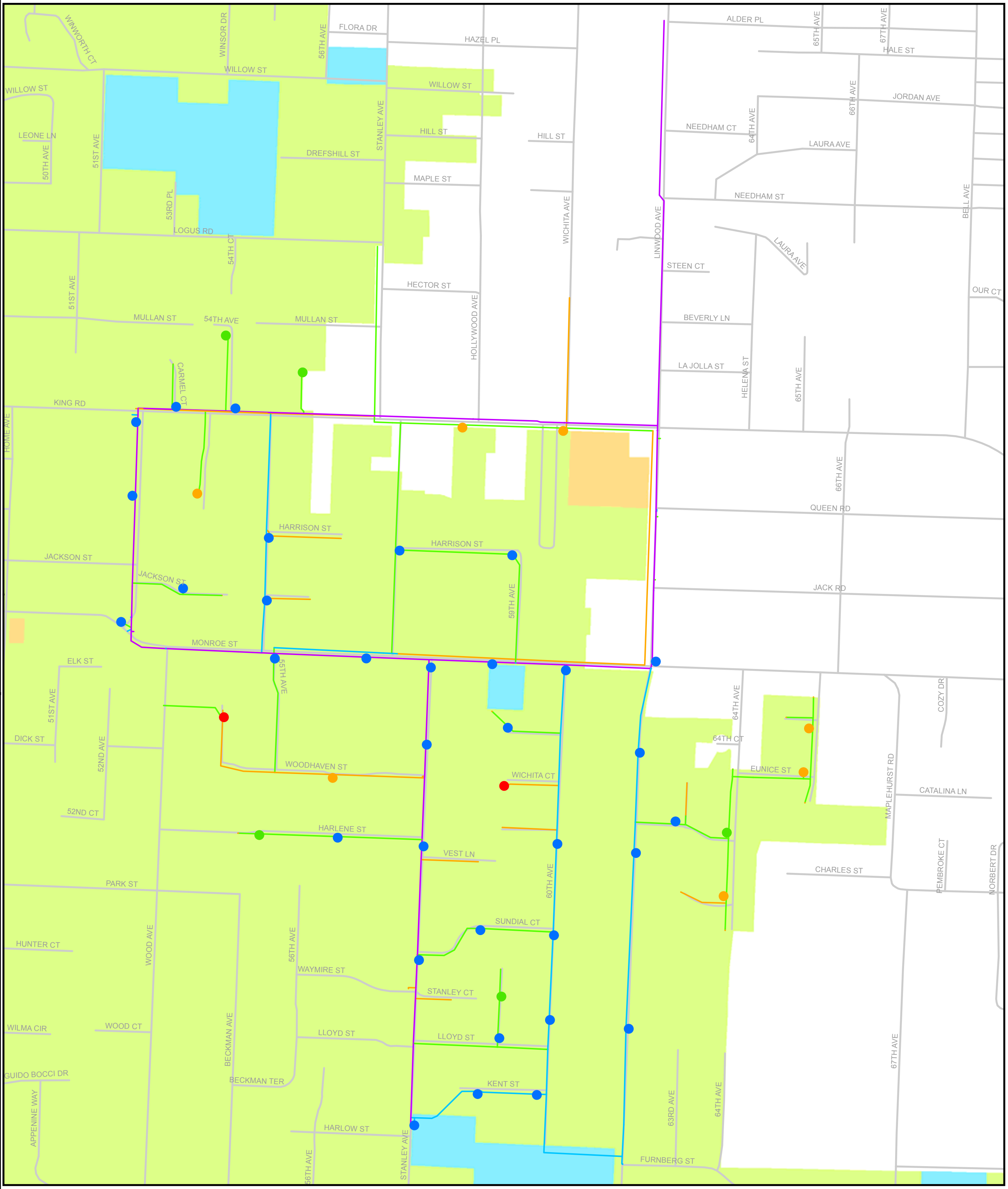
FIGURE 7-20

City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 3
AVAILABLE FIRE FLOW

WEST YOST ASSOCIATES
Consulting Engineers



LEGEND

- Fire Flow < 1,500 gpm
- 1,500 gpm ≤ Fire Flow < 2,500 gpm
- 2,500 gpm ≤ Fire Flow < 3,000 gpm
- Fire Flow ≥ 3,000 gpm
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches

Fire Flow Demand

- 1,500 gpm for 2 hours
- 1,500 gpm for 3 hours
- 2,000 gpm for 4 hours
- 3,000 gpm for 4 hours

Street

N

0 350 700

Scale in Feet

FIGURE 7-21

City of Milwaukie

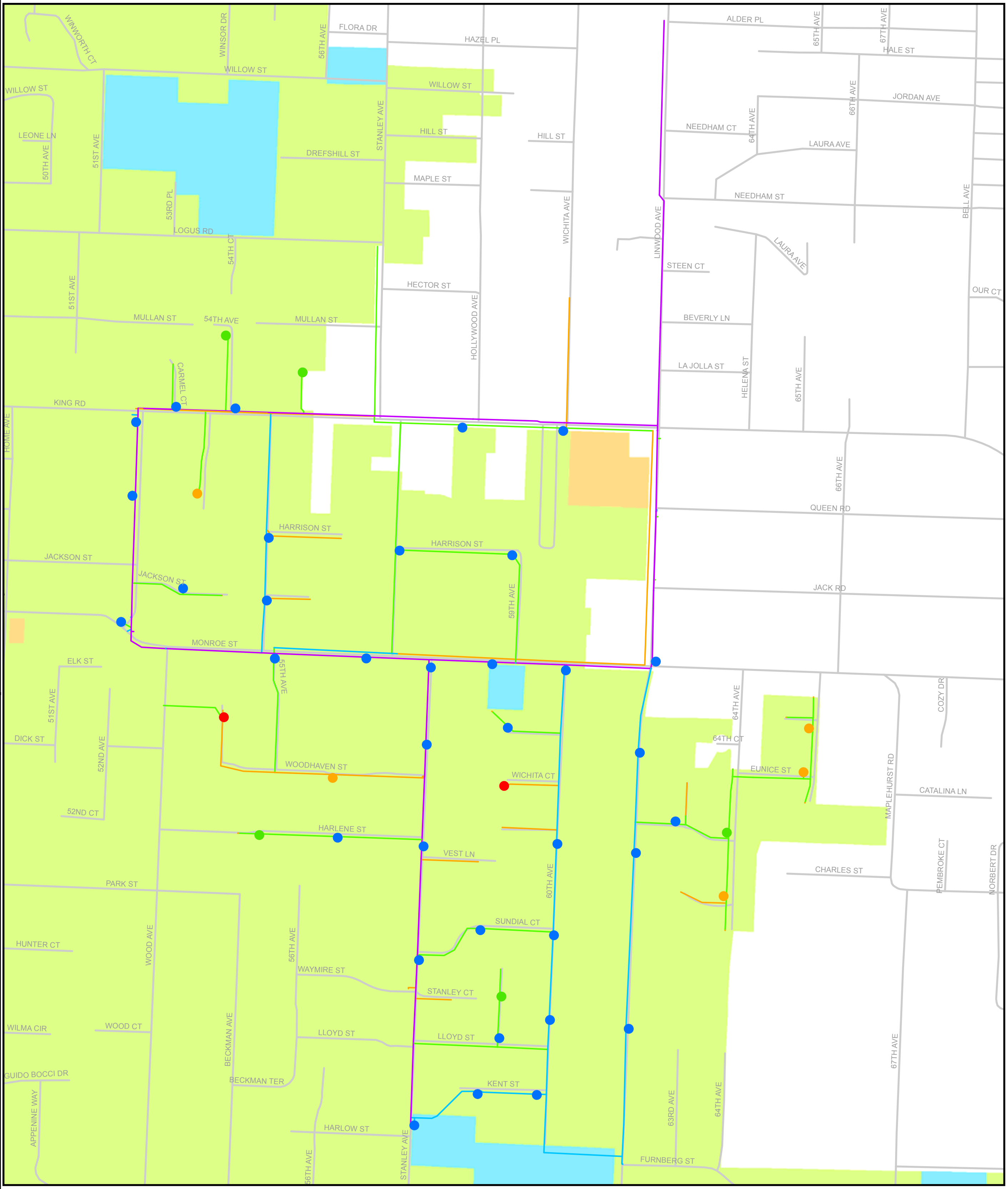
2010 Water System Master Plan

PRESSURE ZONE 3 AVAILABLE FIRE FLOW WITH NEW PUMPS

WEST YOST ASSOCIATES

Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-22_AvailFF_Z3_w_NewPMP_&_PipeUpg.mxd 8/23/2011



LEGEND

- Fire Flow < 1,500 gpm
- 1,500 gpm ≤ Fire Flow < 2,500 gpm
- 2,500 gpm ≤ Fire Flow < 3,000 gpm
- Fire Flow ≥ 3,000 gpm
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches

Fire Flow Demand

- 1,500 gpm for 2 hours
- 1,500 gpm for 3 hours
- 2,000 gpm for 4 hours
- 3,000 gpm for 4 hours

- Street

N

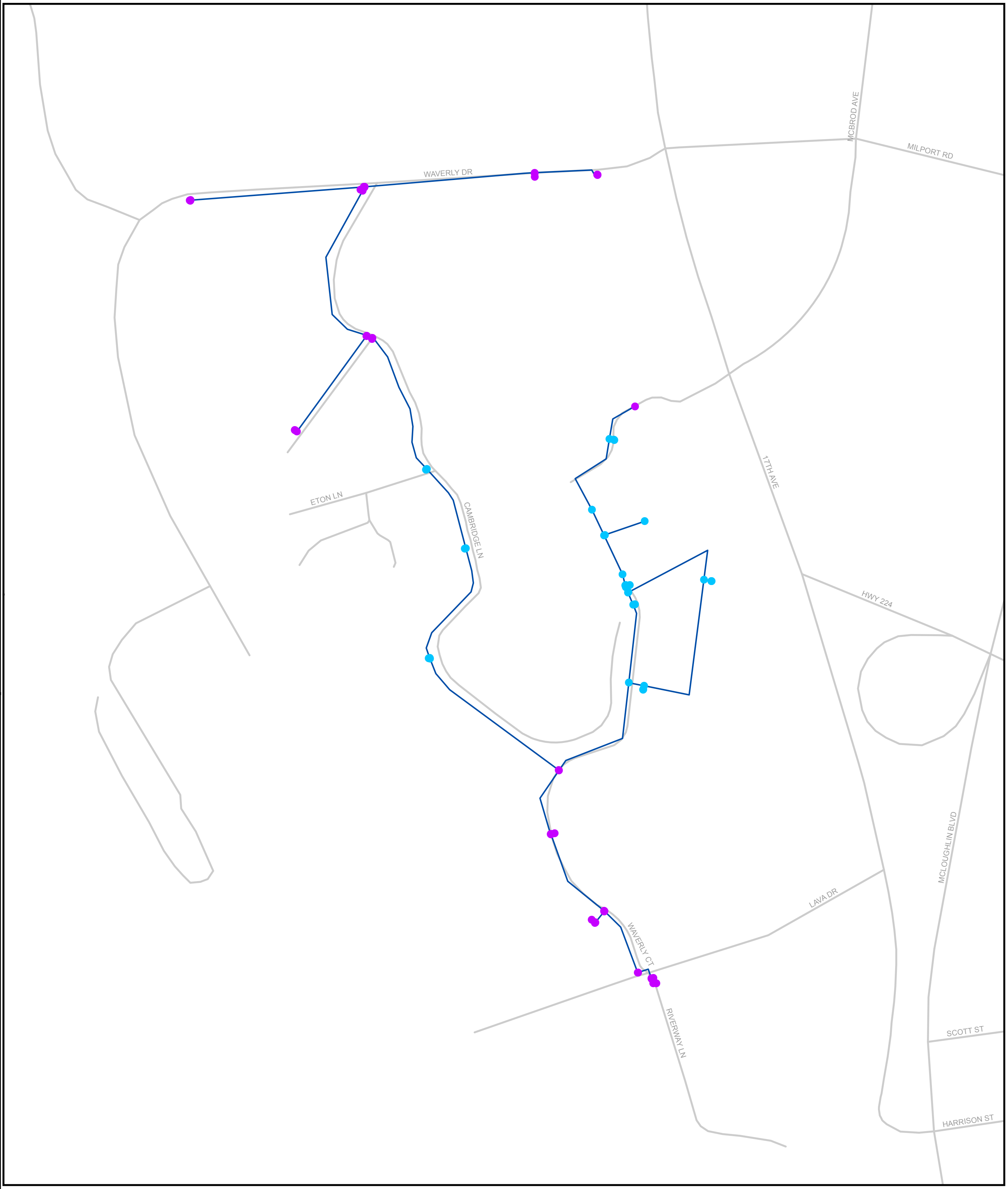
0 350 700

Scale in Feet

FIGURE 7-22
City of Milwaukie
2010 Water System Master Plan
PRESSURE ZONE 3 AVAILABLE FIRE FLOW WITH NEW PUMPS AND PIPELINE UPGRADES

WEST YOST ASSOCIATES
Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-23_PH_Z4.mxd 8/23/2011



LEGEND

- 60 psi ≤ Pressure < 70 psi
- Pressure ≥ 70 psi
- HL/1000 < 10 ft/kft
- HL/1000 ≥ 10 ft/kft
- Street

N

0 200 400

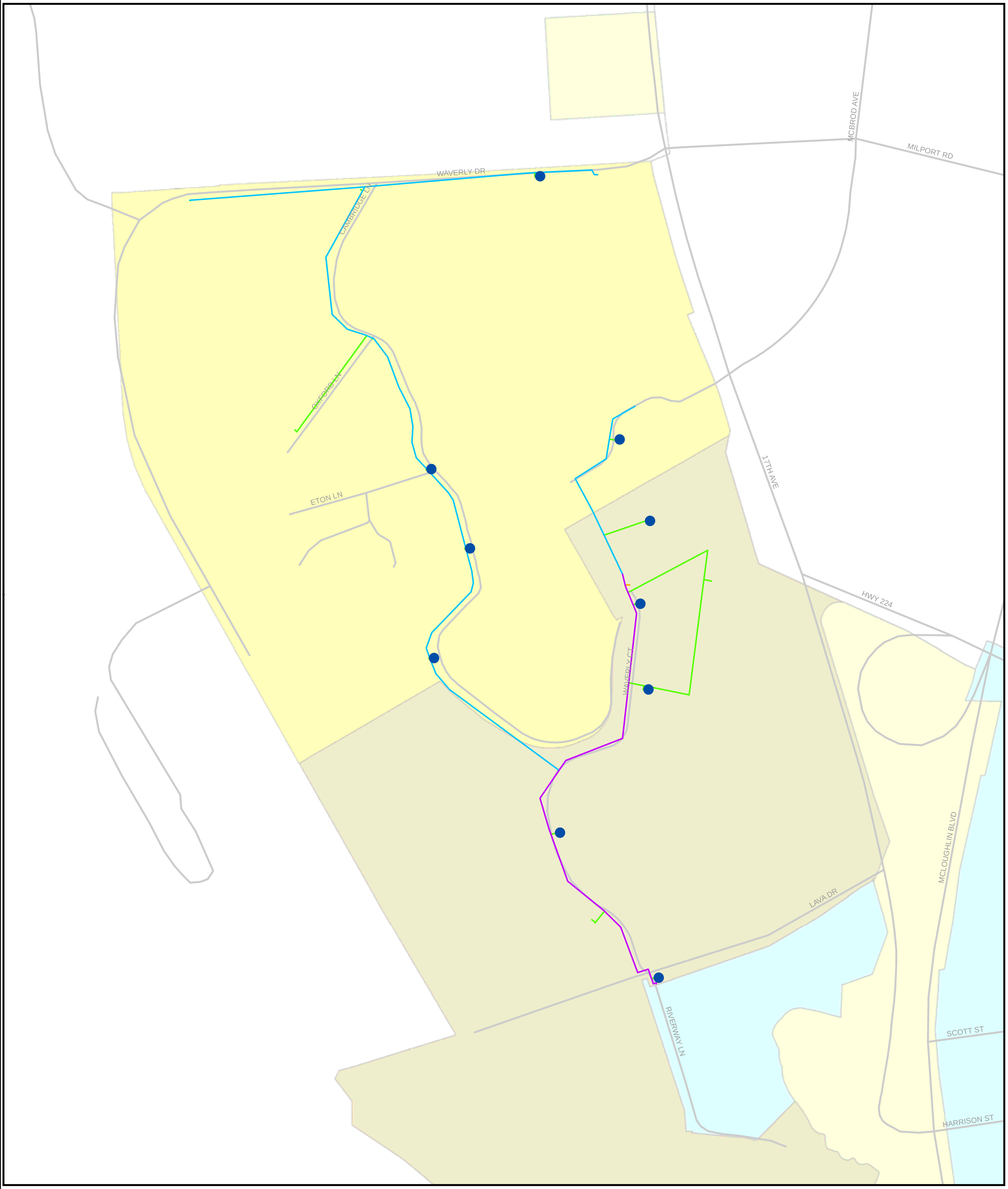
Scale in Feet

FIGURE 7-23

City of Milwaukie
2010 Water System Master Plan
PRESSURE ZONE 4
PEAK HOUR ANALYSIS

WEST YOST ASSOCIATES
Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 7\FIG7-24_MDFF_Z4.mxd 8/23/2011



LEGEND

- Residual Pressure > 20 psi
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches
- Street
- Low Density Residential
- Medium Density Residential
- Moderate Density Residential
- High Density Residential
- Mixed Use
- Commercial
- Industrial
- Public
- Town Center

N

0 200 400

Scale in Feet

FIGURE 7-24

City of Milwaukie
2010 Water System Master Plan
PRESSURE ZONE 4
MAX DAY PLUS FIRE FLOW

WEST YOST ASSOCIATES
Consulting Engineers

CHAPTER 8
Evaluation of Future Water System

This chapter presents an evaluation of the City's future water distribution system (see Figure 8-1) and its capability to meet the recommended performance and planning criteria for the City under future demand conditions. The evaluation includes an analysis of water storage capacity, pumping capacity and the future distribution system's capacity to meet recommended operational and design criteria.

The future system analysis was conducted by West Yost using the City's buildout hydraulic model. In addition to including all of the projected buildout demands, the buildout model also incorporates any improvements identified through the evaluation of the existing system (see Section 7.5, Chapter 7). This allows for any identified deficiency to be associated with the buildout configuration and not the existing system configuration.

The evaluations, findings, and recommendations for addressing the identified water system deficiencies at buildout are organized by pressure zone. A description of the recommended CIP to implement the recommended improvements, including an estimate of construction costs, is provided in Chapter 9.

8.1 FUTURE WATER DEMANDS

8.1.1 City of Milwaukie Buildout

Projected water demands at buildout for the City were calculated by multiplying the recommended unit demands factors (see Table 4-7) by the additional developed acreage projected to occur within the City limit boundary. These vacant parcels are shown on Figure 8-2. Table 8-1 summarizes the additional buildout demands by land use type. Table 8-2 summarizes the additional buildout demands by Pressure Zone.

Table 8-1. Additional Buildout Demand for the City of Milwaukie[a]

City Land Use Category	Additional Acreage to be Developed[a]	Normalized Unit Demand Factor, gpd/acre	Additional Demand, gpd	Additional Demand, gpm
Low Density (LD)	41.0	1,093	44,813	31.1
Moderate Density (MD)	6.9	1,156	7,976	5.5
Medium Density (MED.D)	13.9	1,813	25,201	17.5
High Density (HD)	6.2	865	5,363	3.7
Commercial (C)	0.1	1,317	132	0.1
Mixed Use (C/HD)	2.0	947	1,883	1.3
Industrial (I)	11.9	952	11,329	7.9
Town Center (TC)	8.3	1,372	11,388	7.9
Total	90.3		108,084	75.1

[a] Based on city parcel data provided by the City in Item 001 - Milwaukie Geodatabase. Does not include UAFW, estimated at 11%.

Table 8-2. Additional Buildout Demand by Pressure Zone[(a)]

City Land Use Category	Zone 1		Zone 2		Zone 3		Zone 4	
	Acres[(a)]	Additional Demand, gpm	Acres[(a)]	Additional Demand, gpm	Acres[(a)]	Additional Demand, gpm	Acres[(a)]	Additional Demand, gpm
Low Density (LD)	0.6	0.5	32.1	24.3	5.1	3.8	3.3	2.5
Moderate Density (MD)	3.0	2.4	3.2	2.6	0.7	0.6	0.0	0.0
Medium Density (MED.D)	0.0	0.0	13.9	17.5	0.0	0.0	0.0	0.0
High Density (HD)	0.8	0.5	0.7	0.4	0.0	0.0	4.6	2.8
Commercial (C)	0.0	0.0	0.1	0.1	0.0	0.0	0.0	0.0
Mixed Use (C/HD)	2.0	1.3	0.0	0.0	0.0	0.0	0.0	0.0
Industrial (I)	1.1	0.7	10.9	7.2	0.0	0.0	0.0	0.0
Town Center (TC)	1.1	1.0	7.2	6.9	0.0	0.0	0.0	0.0
Total Vacant Acres:	8.5		68.1		5.8		8.0	
Additional Demand:		6.4		59.0		4.4		5.3

(a) Based on city parcel data provided by the City in Item 001 - Milwaukie Geodatabase. Does not include UAFW, estimated at 11%.

8.1.2 Future Annexation

The UGMA lies within the Metro Regional Urban Growth Boundary (UGB). This is the area outside of the current City limits that is planned for future annexation into the City. The actual timing of annexation for lands within the UGMA is uncertain and will likely proceed on an ad hoc basis. Water demands for buildout of the full UGMA are being evaluated as they relate to the City's ability to supply the area and to help guide policy decisions regarding annexation. Future water demands were projected for the City's UGMA as part of the analysis presented in Chapter 4 and are illustrated in Table 4-8.

Future water demands were projected for Dual Interest Areas A and B. Dual Interest Areas A and B are smaller subsections of the City's UGMA located within Clackamas County. These areas are adjacent to current City limits and have been identified as areas likely to be annexed into the City. As a result, the City must be prepared to provide future water service to these identified Dual Interest Areas while annexation of these areas is anticipated, timing is not yet known. Table 8-3 summarizes the additional demand projected to be added to the system with the annexation of Dual Interest Areas A and B. As illustrated in Table 4-8, the demand for Dual Interest Areas A and B was calculated separately from that of the UGMA demand.

Table 8-3. Future Demand for Dual Interest Areas A and B[(a)]

City Land Use Category	Dual Interest Area A				Dual Interest Area B			
	Acres[(a)]	Normalized Unit Demand Factor, gpd /acre	Additional Demand, gpd	Additional Demand, gpm	Acres[(a)]	Normalized Unit Demand Factor, gpd /acre	Additional Demand, gpd	Additional Demand, gpm
Low Density (LD)	124.7	1,093	136,297	94.7	97.0	1,093	106,021	73.6
Medium Density (MED.D)	2.3	1,813	4,170	2.9	0.0	1,813	0	0.0
Commercial (C)	1.9	1,317	2,502	1.7	0.0	1,317	0	0.0
Industrial (I)	12.8	952	12,186	8.5	0.0	952	0	0.0
Total:	141.7		155,155	107.7	97.0		106,021	73.6

(a) Based on city parcel data provided by the City in Item 001 - Milwaukie Geodatabase Total acreage includes 9.4 and 17.7 acres of vacant acreage for Dual Interest Area A and B respectively. Does not include UAFW, estimated at 11%.

Table 8-4 summarizes the future demand for each of the City's pressure zones. This future demand consists of existing demand, buildout demand and demand from Dual Interest Areas A and B. Based on the service elevations and proximity to the existing City's water system, both Dual Interest Areas will be served by Pressure Zone 2.

Table 8-4. Future Demand for the City

Pressure Zone/Area	Buildout Demand, gpm			Buildout + Dual Interest Areas A and B Demand[(a)], gpm		
	Average Day Demand	Maximum Day Demand[(b)]	Peak Hour Demand[(c)]	Average Day Demand	Maximum Day Demand	Peak Hour Demand
1	324	616	875	324	616	875
2	1,260	2,394	3,402	1,461	2,776	3,945
3	121	230	327	121	230	327
4	52	99	140	52	99	140
Total	1,757	3,339	4,744	1,958	3,721	5,287

(a) Dual Interest Areas A and B will be served by Pressure Zone 2.
(b) Maximum day demand is 1.9 times the average day demand.
(c) Peak hour demand is 2.7 times the average day demand.

8.2 WATER STORAGE CAPACITY

8.2.1 Evaluation

There are two ground-level reservoirs and one elevated storage tank in the City's water system. Together, the storage will be sufficient to meet the City water system's operational, equalization, fire flow, and emergency storage criteria. The volumes required for each of these storage components are presented in Chapter 6 and are summarized below:

- Operational Storage: Because the City's treatment plants and booster pumping stations are capable of operating as long as necessary during the maximum demand period, there is no need for dedicated operational storage within the City's distribution system.
- Equalization Storage: 25 percent of maximum day demand;
- Emergency Storage: 100 percent of average day demand; and
- Fire Flow Storage: Per the Clackamas Fire District #1, fire flow storage is equivalent to the maximum fire flow in the pressure zone multiplied by the required duration. For Zones 1, 2 and 3, a fire flow of 2,500 gpm for a duration of 4 hours (Industrial/Commercial) was assumed for this analysis. This fire flow rate is less than the 3,000 gpm listed in Table 7-6 (City Fire Flow Requirements) in that the 500 gpm for fire sprinklers is not included in the storage calculation. For Zone 4, a fire flow of 1,500 gpm for a duration of 3 hours (Multi-Family Residential) was assumed for this analysis.

Because the City's water supply includes wells, the groundwater basin can account for a portion of the recommended water storage and system peaking capacity in the form of a groundwater credit. The emergency storage credit reflects only groundwater supply which can be reliably accessed when needed (*i.e.*, only wells equipped with auxiliary power). The maximum credit is equal to the required emergency storage capacity.

8.2.2 Results

The existing storage facilities have been evaluated to determine whether they have sufficient capacity to provide the required operational, fire flow, and emergency storage. The City currently has 6.0 MG of water storage as shown in Table 8-5.

8.2.2.1 Buildout

The City has sufficient storage in the system to accommodate buildout demand, as shown in Table 8-5. Although it is desirable for each pressure zone to have its own gravity storage, this is not always feasible, especially with small pressure zones or pressure zones that have inadequate elevation for a storage site. In these cases, sharing storage between pressure zones is allowed provided there is a way to convey the required water into the adjacent pressure zone via pressure reducing valves or pump stations. In the case of a pump station, it is also desirable to provide reliable pumping capacity necessary to deliver the storage provided in the adjacent pressure zone.

Table 8-5. Summary of Storage Requirements: Buildout

Pressure Zone	Storage Facility	Existing Reservoir Capacity, MG	Required Storage Capacity				Potential Groundwater Storage Credits, MG[d]	Total Required Storage, MG	Storage Surplus (Deficiency), MG
			Equalization, MG[a]	Fire Flow, MG[b]	Emergency, MG[c]	Subtotal, MG			
Zone 1	Concrete Reservoir	1.50	0.22	0.60	0.47	1.29	2.17 [d]	0.82	0.68
Zone 2	Elevated Reservoir	1.50	0.86	0.60	1.81	3.28	3.49 [d]	1.46	0.04
Zone 3	Stanley Reservoir	3.00	0.08	0.60	0.17	0.86	0.96 [d]	0.68	2.32
Zone 4	None	0.00	0.04	0.27	0.08	0.38	0.00	0.38	(0.38)
	System Total	6.00					6.62	3.35	

[a] Based on 25 percent of maximum day demand.

[b] Fire flows based on 4 hr duration x 2,500 gpm sprinklered flow for Industrial/Commercial for Zones 1, 2 and 3 and a 3 hr duration x 1,500 gpm Multi-Family Residential for Zone 4.

[c] Based on 100 percent of average day demand.

[d] Groundwater storage credit is equal to 100% of the total pumping capacity for active wells with backup power. Groundwater storage credit can be used to offset required emergency storage only.

As shown in Table 8-5, Zone 4 has a storage deficiency of 0.38 MG. However, Zone 4 has access to Zone 1 storage via the Lava Drive Pump Station. Since Pressure Zone 1 has a storage surplus of 0.68, no additional storage is recommended for Zone 4 at this time.

8.2.2.2 Buildout + Dual Interest Areas

As shown in Table 8-6, the City has sufficient storage to provide demand at buildout plus the addition of Dual Interest Areas A and B. Zone 2 has a storage deficiency of 0.1 MG. Zone 2 has access to Zone 1 storage via the W2 pump station, and to Zone 3 storage via the W6 Pump Station. Since Zones 1 and 3 have a combined storage surplus of 2.98 MG, no additional storage is recommended for Zone 2 at this time. Zone 4 has a storage deficiency of 0.38 MG; however, as explained above, Zone 4 has access to Zone 1 storage via the Lava Drive Pump Station. Even with Zone 1 supplementing the storage deficiency of Zone 2 (0.1 MG), there is enough surplus storage in Zone 1 (0.68 MG) to also supplement the Zone 4 storage deficiency (0.38 MG). As a result, no additional storage is required for Zone 4 at this time.

8.2.2.3 Urban Growth Management Area

As outlined in Chapter 4, addition of the UGMA, excluding the Dual Interest Areas, to the City's water system would increase water demand by approximately 4.5 mgd; nearly tripling the City's current water demand. Calculating the increase in storage necessary for the City to provide water service to the UGMA requires detailed analysis of the UGMA area. Such an analysis includes establishing the location of pressure zones, booster pump stations, and groundwater wells to increase supply and required fire flows.

It is beyond the scope of this Water System Master Plan to calculate the storage required to provide water service to the UGMA. Generally speaking, providing service to the UGMA would require a significant increase in storage. As shown in Table 8-6, there is a remaining storage surplus of approximately 40 percent in the City water system, assuming buildout and incorporation of Dual Interest Areas A and B. Even if all of the City's storage surplus could be applied to the UGMA, the City would still have to make a significant investment to construct additional water storage to serve the nearly 250 percent increase in water demand. This future water storage capacity could be increased by the construction of additional storage facilities as well as the addition of new ground water wells, providing an increase in the City's available groundwater storage credit.

Table 8-6. Summary of Storage Requirements: Buildout + Dual Interest Areas A and B

Pressure Zone	Storage Facility	Existing Reservoir Capacity, MG	Required Storage Capacity				Potential Groundwater Storage Credits, MG[d]	Total Required Storage, MG	Storage Surplus (Deficiency), MG
			Equalization, MG[a]	Fire Flow, MG[b]	Emergency, MG[c]	Subtotal, MG			
Zone 1	Concrete Reservoir	1.50	0.22	0.60	0.47	1.29	2.17 [d]	0.82	0.68
Zone 2	Elevated Reservoir	1.50	1.00	0.60	2.10	3.70	3.49 [d]	1.60	(0.10)
Zone 3	Stanley Reservoir	3.00	0.08	0.60	0.17	0.86	0.96 [d]	0.68	2.32
Zone 4	None	0.00	0.04	0.27	0.08	0.38	0.00	0.38	(0.38)
	System Total:	6.00					6.62	3.48	

[a] Based on 25 percent of maximum day demand.

[b] Fire flows based on 4 hr duration x 2,500 gpm sprinklered flow for Industrial/Commercial for Zones 1, 2 and 3 and a 3 hr duration x 1,500 gpm Multi-Family Residential for Zone 4.

[c] Based on 100 percent of average day demand.

[d] Groundwater storage credit is equal to 100% of the total pumping capacity for active wells with backup power. Groundwater storage credit can be used to offset required emergency storage capacity only.

8.3 PUMPING CAPACITY

8.3.1 Groundwater Pumping Capacity

8.3.1.1 Evaluation

The City's pumping capacity was evaluated to assess its ability to deliver a reliable firm capacity to the existing service area. AWWA Manual 31 "Distribution System Requirements for Fire Protection" suggests that a standby pump and reliable power be provided to each pump station. Since a standby pump for groundwater well pumps is not practical, the firm groundwater pumping capacity is defined for the City's groundwater wells as the total well capacity for all wells that can be accessed during a power outage, or all wells with back-up power. All the City's groundwater well pump stations are equipped with back-up power; therefore, the groundwater pumping capacity in the City is equal to the total well capacity.

In addition, AWWA suggests that the pumping capacity criterion for the water system be sufficient to meet maximum day demand within the service area, assuming that gravity storage is available. If there is no gravity storage available within the service area, the total pumping capacity must be equivalent to the larger design demand, which in the City's case is either maximum day demand plus fire flow or peak hour demand, whichever is greater.

8.3.1.2 Results

The City has gravity storage available within the service area. As a result, the pumping capacity for the water system must be sufficient to meet the maximum day demand. The pumping capacity analysis indicates that the City's existing, firm groundwater pumping capacity meets the pumping capacity criterion for the entire service area during a maximum day demand condition. Table 8-7 summarizes the pumping capacity of each pump station. As shown in Table 8-8, the City has a pumping capacity surplus of 1,266 gpm for the Buildout System. The Buildout plus the Dual Interest Areas A and B system has a pumping capacity surplus of 884 gpm.

As outlined in Chapter 4, addition of the UGMA, excluding the Dual Interest Areas, to the City's water system would increase water demand by approximately 4.5 mgd; nearly tripling the City's current water demand. Because of this sizeable increase, the City would have to make a significant investment to construct additional wells to increase supply and pumping facilities to distribute the water to the UGMA. As shown in Table 8-8, by providing water service to the UGMA, the City would have a pumping capacity deficiency of 4,631 gpm. The City's future groundwater pumping capacity could be increased by the construction of additional groundwater wells, which would require increasing water rights. For example, the groundwater pumping capacity deficiency could be corrected with the addition of ten new 500 gpm groundwater wells.

Table 8-7. Summary of Existing Pumping Facilities

Pump Station	Backup Power	Status	Pump 1[(a)], gpm	Firm Capacity[(b)], gpm	City Water Rights[(c)], gpm
Well 02	✓	Active	394	394	557
Well 03	✓	Active	511	511	585
Well 04	✓	Active	605	605	538
Well 05	✓	Active	605	605	718
Well 06	✓	Active	670	670	809
Well 07	✓	Active	1,120	1,120	1,198
Well 08	✓	Active	700	700	727
Total				4,605	

(a) Pump flow data provided by the City of Milwaukie in Data Request Item 16.
(b) For groundwater well pumps, the firm groundwater pumping capacity is defined as the total well capacity for all wells that can be accessed during a power outage, or all wells with backup power.
(c) Based on data provided by City in Item 014.

Table 8-8. Evaluation of Total Firm Pumping Capacity and Maximum Day Demand

Pump Station	Existing Firm Pumping Capacity, gpm	City Water Rights[(a)], gpm	Buildout Maximum Day Demand[(b)], gpm	Buildout + Dual Interest Areas A and B Maximum Day Demand, gpm	UGMA, Maximum Day Demand (deficiency), gpm
Groundwater Wells	4,605[(c)]	5,132	3,339	3,721	9,236
Pumping Capacity Surplus, gpm			1,266	884	(4,631)

(a) Based on data provided by City in Item 014.
(b) Maximum day demand is 1.9 times the average day demand.
(c) Defined as the total active well capacity for all wells that can be accessed during a power outage, or all wells with backup power.

8.3.2 Distribution Pumping Capacity

8.3.2.1 Evaluation

This evaluation was conducted assuming all recommendations made in the existing system chapter (Chapter 7) have been implemented. The City's water system must be capable of providing the required peak hour demand or maximum day plus fire flow demand to each pressure zone. The distribution pump firm capacity of each pressure zone is defined as the pumping capacity of each pump station serving the pressure zone with the largest pump out of service. If a pump station has a single pump with a backup generator, the pump capacity is included in the firm capacity. However, if the pressure zone has gravity storage, the required distribution pump firm capacity can be reduced to equal the maximum day demand of the pressure zone. Each pressure zone was analyzed individually taking into consideration that all pressure zones must meet the requirements at the same time. Tables 8-9 and 8-10 summarize the available capacity by pressure zone for the two future scenarios.

8.3.2.2 Results

Both Zone 1 and Zone 2 have access to gravity storage. As a result, the distribution pump firm capacity must be capable of providing the maximum day demand of the pressure zone. As shown in Table 8-9 and Table 8-10, the firm capacity of Zone 1 exceeds the maximum day demand with addition of the Buildout and Dual Interest Areas A and B scenarios. As a result, the pumping capacity is sufficient to serve Zone 1 future development conditions.

The firm capacity of Zone 2 is less than the maximum day demand under both the Buildout and Dual Interest Areas A and B scenarios. However, the hydraulic model indicates that approximately 640 gpm can be supplied from Zone 1 through the W2 pump station and 500 gpm can be supplied from Zone 3 through the W6 pump station during maximum day demand conditions. With this additional pumping from Zone 1 and Zone 3, the resulting pumping capacity is sufficient to serve Zone 2 future development conditions.

Both Zone 3 and Zone 4 do not have access to gravity storage. As a result, the distribution pump firm capacity must be equal to the larger of the required peak hour demand or maximum day plus fire flow demand. The maximum day plus fire flow demand is greater than the peak hour demand in both zones. As a result, the distribution firm capacity of Zone 3 and Zone 4 must be equal to the maximum day plus fire flow demand.

As discussed in Chapter 7, two 1,750 gpm pumps are recommended to be installed in Zone 3 to meet existing pumping deficiencies in Zone 3. Assuming that these improvements have been made, Table 8-9 and Table 8-10 confirm that the firm capacity of Zone 3 exceeds the maximum day plus fire flow demand with addition of the Buildout and Dual Interest Areas A and B scenarios. As a result, the pumping capacity is sufficient to serve Zone 3 future development conditions.

The firm capacity of Zone 4 exceeds the maximum day plus fire flow demand with addition of the Buildout and Dual Interest Areas A and B scenarios. As a result, the pumping capacity of Zone 4 is sufficient to serve Zone 4 future development.

Table 8-9. Summary of Pumping Capacity Requirements: Buildout

Service Area	Pump Station	Total Capacity, gpm	Firm Capacity, gpm[a]	Existing Backup Power	Future Average Day Demand, gpm	Future Maximum Day Demand, gpm[b]	Required Fire Flow, gpm	Future Maximum Day Demand plus Fire Flow, gpm	Future Peak Hour Demand, gpm[c]	Required Additional Firm Pumping Capacity, gpm
Zone 1	Treatment Plant 235 No. 1	700	1,400	Diesel Generator	324	616	2,500	3,116	875	0[d]
	Treatment Plant 235 No. 2	700		Diesel Generator						
	Treatment Plant 235 No. 3	700		Diesel Generator						
	Subtotal	2,100	1,400							
Zone 2	Treatment Plant 47 No. 1	900	900	Diesel Generator	1,260	2,394	2,500	4,894	3,402	0[e]
	Treatment Plant 47 No. 2	900		Diesel Generator						
	Well 8	700	700	Diesel Generator						
	Subtotal	2,500	1,600							
Zone 3	Zone 3 Pump No. 1	200	3,350	Diesel Generator	121	230	2,500	2,730	327	0[f]
	Zone 3 Pump No. 2	200		Diesel Generator						
	Zone 3 Pump No. 3	600		Diesel Generator						
	Zone 3 Pump No. 4	600		Diesel Generator						
	Zone 3 High Demand Pump 1	1,750		Diesel Generator						
	Zone 3 High Demand Pump 2	1,750		Diesel Generator						
	Subtotal	5,100	3,350							
Zone 4	Lava Pump 1	300	2,350	Portable Diesel Generator	52	99	1,500	1,599	140	0[g]
	Lava Pump 2	300		Portable Diesel Generator						
	Lava Pump 3	1,750		Portable Diesel Generator						
	Lava Pump 4	1,750		Portable Diesel Generator						
	Subtotal	4,100	2,350							
	Total Pumping Capacity, gpm	13,800	8,700		1,757	3,338		12,338	4,744	0
	Total Pumping Capacity, mgd	19.9	12.5		2.5	4.8		17.8	6.8	0.0

(a) Firm capacity for booster pump stations is defined as pumping capacity with the largest pump out of service and/or single pump with backup generator. Available groundwater pumping capacity is defined as the pumping capacity of all wells that can be accessed during a power outage, or all wells with backup power.

(b) Maximum day demand equals 1.9 times average day demand.

(c) Peak hour demand equals 2.7 times average day demand.

(d) To provide a more conservative estimate of pumping capacity, the Treatment Plant 235 pumps supplying Zone 1 were used in this calculation. The available pumping capacity of the wells serving this zone is equal to 1,510 gpm. Since this zone has access to gravity storage, the pumping capacity is required to be equal to the maximum day demand for the pressure zone.

(e) To provide a more conservative estimate of pumping capacity, the Treatment Plant 47 pumps supplying Zone 2 as well as well 8 were used in this calculation. The available pumping capacity of the wells serving this zone is equal to 2,425 gpm. Since this zone has access to gravity storage, the pumping capacity is required to be equal to the maximum day demand for the pressure zone. The hydraulic model indicates that approximately 435 gpm can be supplied from Zone 1 through the W2 pump station and approximately 361 gpm can be supplied from Zone 3 through the W6 pump station during maximum day demand conditions.
Based on this, the additional pumping capacity required is Maximum Day Demand (2,396 gpm) – Firm Pumping Capacity (1,600 gpm) – W2 pump station (435 gpm) - W6 pump station (361 gpm) = 0 gpm.

(f) Because all water supplied to this zone is pumped from the Stanley Reservoir by the Zone 3 booster pumps, the pumping capacity of the booster pump station was used in this calculation. Because this zone does not have access to gravity storage, the pumping capacity for the pressure zone is required to be equal to the maximum day demand plus fire flow or peak hour demand, whichever is greater.

(g) Because all water supplied to this zone is pumped from Zone 1, the pumping capacity of Lava Drive Pump Station was used in this calculation. Because this zone does not have access to gravity storage, the pumping capacity for the pressure zone is required to be equal to the maximum day demand plus fire flow or peak hour demand, whichever is greater.

(h) Highlighted cell refers to Demand Condition that applies to the Zone.

Table 8-10. Summary of Pumping Capacity Requirements: Buildout + Dual Interest Areas A and B

Service Area	Pump Station	Total Capacity, gpm	Firm Capacity, gpm[a]	Existing Backup Power	Future Average Day Demand, gpm	Future Maximum Day Demand, gpm[b]	Required Fire Flow, gpm	Future Maximum Day Demand plus Fire Flow, gpm	Future Peak Hour Demand, gpm[c]	Required Additional Firm Pumping Capacity, gpm
Zone 1	Treatment Plant 235 No. 1	700	1,400	Diesel Generator	324	616	2,500	3,116	875	0[d]
	Treatment Plant 235 No. 2	700		Diesel Generator						
	Treatment Plant 235 No. 3	700		Diesel Generator						
	Subtotal	2,100	1,400							
Zone 2	Treatment Plant 47 No. 1	900	900	Diesel Generator	1,461	2,776	2,500	5,276	3,945	0[e]
	Treatment Plant 47 No. 2	900		Diesel Generator						
	Well 8	700	700	Diesel Generator						
	Subtotal	2,500	1,600							
Zone 3	Zone 3 Pump No. 1	200	3,350	Diesel Generator	121	230	2,500	2,730	327	0[f]
	Zone 3 Pump No. 2	200		Diesel Generator						
	Zone 3 Pump No. 3	600		Diesel Generator						
	Zone 3 Pump No. 4	600		Diesel Generator						
	Zone 3 High Demand Pump 1	1,750								
	Zone 3 High Demand Pump 2	1,750								
	Subtotal	5,100	3,350							
Zone 4	Lava Pump 1	300	2,350	Portable Diesel Generator	52	99	1,500	1,599	140	0[g]
	Lava Pump 2	300		Portable Diesel Generator						
	Lava Pump 3	1,750		Portable Diesel Generator						
	Lava Pump 4	1,750		Portable Diesel Generator						
	Subtotal	4,100	2,350							
	Total Pumping Capacity, gpm	13,800	8,700		1,958	3,720		12,720	5,287	0
	Total Pumping Capacity, mgd	19.9	12.5		2.8	5.4		18.3	7.6	0.0

(a) Firm capacity for booster pump stations is defined as pumping capacity with the largest pump out of service and/or single pump with backup generator. Available groundwater pumping capacity is defined as the pumping capacity of all wells that can be accessed during a power outage, or all wells with backup power.

(b) Maximum day demand equals 1.9 times average day demand.

(c) Peak hour demand equals 2.7 times average day demand.

(d) To provide a more conservative estimate of pumping capacity, the Treatment Plant 235 pumps supplying Zone 1 were used in this calculation. The available pumping capacity of the wells serving this zone is equal to 1,510 gpm. Since this zone has access to gravity storage, the pumping capacity is required to be equal to the maximum day demand for the pressure zone.

(e) To provide a more conservative estimate of pumping capacity, the Treatment Plant 47 pumps supplying Zone 2 as well as well 8 were used in this calculation. The available pumping capacity of the wells serving this zone is equal to 2,425 gpm. Since this zone has access to gravity storage, the pumping capacity is required to be equal to the maximum day demand for the pressure zone. The hydraulic model indicates that aproximately 640 gpm can be supplied from Zone 1 through the W2 pump station and approximately 500 gpm can be supplied from Zone 3 through the W6 pump station during maximum day demand conditions. Based on this, the additional pumping capacity required is Maximum Day Demand (2,740 gpm) – Firm Pumping Capacity (1,600 gpm) – W2 pump station (640 gpm) - W6 (500gpm) = 0 gpm.

(f) Because all water supplied to this zone is pumped from the Stanley Reservoir by the Zone 3 booster pumps, the pumping capacity of the booster pump station was used in this calculation. Because this zone does not have access to gravity storage, the pumping capacity for the pressure zone is required to be equal to the maximum day demand plus fire flow or peak hour demand, whichever is greater.

(g) Because all water supplied to this zone is pumped from Zone 1, the pumping capacity of Lava Drive Pump Station was used in this calculation. Because this zone does not have access to gravity storage, the pumping capacity for the pressure zone is required to be equal to the maximum day demand plus fire flow or peak hour demand, whichever is greater.

(h) Highlighted cell refers to Demand Condition that applies to the Zone.

8.3.3 Critical Pumping Facilities

All critical pumping facilities should be equipped with an on-site, stand-by power generator. Critical pumping facilities are defined as those facilities that provide service to pressure zones and/or service areas without sufficient emergency storage and that meet the following criteria:

- The largest facility that provides water to a particular pressure zone and/or service area;
- A facility that provides the sole source of water to single or multiple pressure zones and/or service area(s);
- A facility that provides water from key groundwater supply wells (depends on capacity, quality, and location) into a pressure zone and /or service area.

All wells, pump stations and treatment facilities are equipped with back-up power generators to provide pumping capacity during a power outage. The Lava Drive Booster Pump Station lacks an on-site generator. Due to space constraints at the Lava Drive Booster Pump Station, serving Zone 4, a dedicated on-site back-up generator was not constructed. A portable generator is stored at the Milwaukie Johnson Creek facility and is dedicated for emergency use only at the Lava Drive Booster Pump Station.

8.4 WATER DISTRIBUTION SYSTEM

8.4.1 Methodology

A steady state hydraulic analysis using the model, as discussed in Chapter 6, was conducted to identify areas of the existing water distribution system that do not meet the recommended system performance criteria. The results of the evaluation of the water distribution system are presented below for the following existing demand scenarios:

- Peak Hour Demand—peak hour demands are met by either flows from the storage reservoirs or from the pump stations. A peak hour flow condition was simulated for the existing distribution facilities to evaluate their capability to meet this peak hour demand condition.
- Maximum Day Demand plus Fire Flow—to evaluate the system under the maximum day demand plus fire flow condition, a two-step analysis was performed. The first step used the InfoWater's "Available Fire Flow Analysis" option to determine if the minimum pressure and required fire flow could be met with the existing water facilities. Fire flow demands were assigned by land use type and simulated at existing hydrant locations in each pressure zone. If the analysis indicated that the system failed to meet the minimum requirements for pressure and flow, a second analysis was performed. The second analysis involved running the model with pipeline improvements/system modifications added to the distribution system to eliminate previously identified deficiencies.

As shown in Table 8-4, the City's existing service area peak hour demand was calculated to be 4,744 gpm for the buildout system and 5,287 for the buildout system plus Dual Interest Areas A and B. This peak hour demand represents a peaking factor of 2.7 times the average day demand. During a peak hour condition, a minimum pressure of 40 psi must be maintained throughout the system. Maximum head loss per thousand feet of distribution main generally should not exceed 10 ft/kft and maximum velocities should not exceed 7 fps. Details of system pressures in each of the City's pressure zones as simulated in the model under the peak hour demand condition are discussed below.

Fire flow demands were simulated at various locations within the City's service area to determine whether or not the minimum residual pressure criterion of 20 psi within the pressure zone could be maintained during a maximum day demand condition with a concurrent fire flow. Table 8-11 presents fire flow demand requirements based on land use categories. This fire flow data is also illustrated on Figure 8-3.

Table 8-11. City Fire Flow Demand Requirements[a]

Land Use	Fire Flow, gpm	Duration, hours
Single-Family Residential[b]	1,500	2
Multi-Family Residential	1,500	3
Institutional[c]	2,000[d]	4
Industrial/Commercial[e]	3,000[d]	4

(a) Specific fire flows were determined from Table B105.1 of the 2007 OFC, and depend on construction type and building square footage (fire area). The fire flow requirements for Institutional and Industrial/Commercial buildings assume these building types are fully sprinkled.
(b) Single-Family includes Low Density Residential, Medium Density Residential and Moderate Density Residential land use.
(c) Institutional includes Public and Town Center land uses.
(d) Includes a 500 gpm demand for on-site sprinkler flow.
(e) Industrial/Commercial includes Commercial, Mixed Use and, Industrial land uses.

Pipelines are typically designed to deliver peak hour flows and maximum day demands plus fire flows within acceptable pressure, velocity, and head loss ranges as stated in Chapter 6.

8.4.2 Evaluation and Results

The following addresses the results of the peak hour demand and maximum day demand plus fire flow analyses by individual pressure zone for the Buildout plus Dual Interest Areas A and B system.

8.4.2.1 Pressure Zone 1: Buildout System plus Dual Interest Areas A and B

Because Dual Interest Areas A and B are not served by Pressure Zone 1, there was no difference in the analysis results between the buildout and buildout plus Dual Interest Areas A and B systems. The analysis for Pressure Zone 1 was conducted assuming all existing system recommendations made in Chapter 7 have been implemented.

8.4.2.1.1 Peak Hour

During a peak hour demand condition, results indicate that the existing infrastructure and distribution system in Pressure Zone 1 can adequately deliver peak hour demands to most of the Zone under the City's minimum pressure criteria of 40 psi (see Figure 8-4). System pressures in the zone range from 36 to 74 psi. The locations with pressures below 40 psi are within 5 psi of the acceptable range, so no mitigation is recommended at this time. As illustrated on Figure 8-4, essentially all of the pipelines in Pressure Zone 1 meet the City's head loss criteria of 10 ft/kft.

There were several short segments of pipeline that exceeded the recommended head loss criteria. After discussions with the City, it was agreed that these segments are located at non-critical locations, thus no mitigation is recommended at this time.

8.4.2.1.2 Maximum Day plus Fire Flow Demand

Fire flows were simulated at hydrant locations throughout Zone 1 based on land use. The fire flow simulation results show that many areas in Pressure Zone 1 cannot maintain a minimum system pressure of 20 psi under the required fire flow. The results of this analysis are shown on Figures 8-5 and 8-6. Figure 8-6 presents the available fire flow at a 20 psi residual throughout the Pressure Zone 1 system. Thirteen (13) hydrants could not meet the minimum flow requirement. This represents approximately eight percent of the modeled hydrants within the pressure zone. Approximately two percent of the modeled hydrants in Pressure Zone 1 were unable to provide a minimum fire flow of 1,500 gpm at the required residual pressure of 20 psi. However, when the fire flow demand was split between two hydrants, the required residual pressure of 20 psi could be met. Because it was assumed that all recommended existing system improvements have been implemented, these results were an improvement over those in Chapter 7.

8.4.2.2 Pressure Zone 2: Buildout System

The analysis for Pressure Zone 2 was conducted assuming all existing system recommendations made in Chapter 7 have been implemented.

8.4.2.2.1 Peak Hour

The results of the peak hour demand analysis by including the Buildout scenario did not differ significantly from the existing system analysis performed in Chapter 7. The results indicate that the existing infrastructure and distribution system in Pressure Zone 2 is adequate to deliver peak hour demands under the City's minimum pressure criteria of 40 psi (see Figure 8-7). Also, essentially all of the pipelines in Pressure Zone 2 meet the City's head loss criteria of 10 ft/kft, as illustrated in Figure 8-7.

8.4.2.2.2 Maximum Day plus Fire Flow Demand

Fire flows were simulated at hydrant locations throughout Zone 2 based on land use. The fire flow simulation results show that many areas in Pressure Zone 2 cannot maintain a minimum system pressure of 20 psi under the required fire flow. The results of this analysis are shown on Figures 8-8 and 8-9. Figure 8-9 presents the available fire flow at a 20 psi residual throughout Pressure Zone 2. Twenty-eight (28) hydrants of the 504 hydrants modeled could not meet the minimum flow requirement. This represents approximately six percent of the hydrants within the pressure zone. Approximately five percent of the modeled hydrants in Pressure Zone 2 were unable to provide a minimum fire flow of 1,500 gpm at the required residual pressure of 20 psi. However, when the fire flow demand was split between two hydrants, the required residual pressure of 20 psi could be met. Because it was assumed that all recommended existing system improvements have been implemented, these results were an improvement over those in Chapter 7.

8.4.2.3 Pressure Zone 2: Buildout plus Dual Interest Areas A and B System

The analysis for Pressure Zone 2 was generally conducted assuming all existing system recommendations made in Chapter 7 have been implemented. The peak hour demand analysis was also completed without the improvements identified in Chapter 7.

8.4.2.3.1 Peak Hour

The results of the peak hour demand analysis by including the Buildout plus Dual Interest Areas A and B scenario did not differ significantly from the existing system analysis performed in Chapter 7. The results indicate that the buildout system plus Dual Interest Areas A and B in Pressure Zone 2 is adequate to deliver peak hour demands under the City's minimum pressure criteria of 40 psi (see Figure 8-10). This is the case even if the improvements recommended in Chapter 7 are not included. Also the majority of the pipelines in Pressure Zone 2 meet the City's head loss criteria of 10 ft/kft, as illustrated in Figure 8-10.

8.4.2.3.2 Maximum Day plus Fire Flow Demand

Fire flows were simulated at hydrant locations throughout Zone 2 based on land use. The fire flow simulation results show that many areas in Pressure Zone 2 cannot maintain a minimum system pressure of 20 psi under the required fire flow. The results of this analysis are shown on Figures 8-11 and 8-12. Figure 8-12 presents the available fire flow at a 20 psi residual throughout Pressure Zone 2. Thirty-four (34) hydrants of the 504 hydrants modeled could not meet the minimum flow requirement. This represents approximately seven percent of the hydrants within the pressure zone. Approximately five percent of the modeled hydrants in Pressure Zone 2 were unable to provide a minimum fire flow of 1,500 gpm at the required residual pressure of 20 psi. However, when the fire flow demand was split between two hydrants, the required residual pressure of 20 psi could be met. Because it was assumed that all recommended existing system improvements have been implemented, these results were an improvement over those in Chapter 7.

Dual Interest Areas A and B will be annexed into Pressure Zone 2, the following recommended improvements are required for the future system in Pressure Zone 2.

Dual Interest Area A

- To serve Dual Interest Area A, CRW has an 12-inch diameter water line in SE Linnwood Avenue, 8-inch and a 6-inch diameter pipelines in SE Hollywood Avenue that form a loop with the rest of their system. Without additional analysis of their system, it is not clear whether CRW would be willing to transfer ownership of these lines to the City when the area is annexed to the City. New 8-inch lines are included at this time to serve the area. West Yost has assumed installation of approximately 6,060 linear feet of 8-inch diameter DI pipeline as shown on Figure 8-13 to provide backbone infrastructure to this new area.

Dual Interest Area B

- To serve Dual Interest Area B, CRW has an 8-inch diameter water line in SE Lake Road and a 6-inch diameter line in SE Kuehn Road that form a loop with the rest of their system. Without additional analysis of their system, it is not clear whether CRW would be willing to transfer ownership of these lines to the City when the area is annexed to the City. New 8-inch lines are included at this time to serve the area. West Yost has assumed installation of approximately 4,570 linear feet of 8-inch diameter DI pipeline as shown on Figure 8-14 to provide backbone infrastructure to this new area.

8.4.2.4 Pressure Zone 3: Buildout System plus Dual Interest Areas A and B

The analysis for Pressure Zone 3 was conducted assuming all existing system recommendations made in Chapter 7 have been implemented.

Because Dual Interest Areas A and B are not included in Pressure Zone 3, there was no difference in the analysis results between the buildout and the buildout plus Dual Interest Areas A and B systems.

8.4.2.4.1 Peak Hour

The results of the peak hour demand analysis by including the Buildout scenario did not differ significantly from the existing system analysis performed in Chapter 7. The results indicate that the future system in Pressure Zone 3 could adequately deliver peak hour demands under the City's minimum pressure criteria of 40 psi (see Figure 8-15). Head losses and velocities in Pressure Zone 3 meet the City maximum design criteria of 10 ft/kft and 7 fps.

8.4.2.4.1.1 Maximum Day plus Fire Flow Demand

Fire flows were simulated at hydrant locations throughout Zone 3based on land use type. Results indicate that the existing system in Pressure Zone 3 is able to meet the required residual pressure of 20 psi under maximum day plus fire flow demand conditions (see Figure 8-16).

8.4.2.5 Pressure Zone 4: Buildout System plus Dual Interest Areas A and B

The analysis for Pressure Zone 4 was conducted assuming all existing system recommendations made in Chapter 7 have been implemented.

Because Dual Interest Areas A and B are not included in Pressure Zone 3, there was no difference in the analysis results between the buildout and the buildout plus Dual Interest Areas A and B systems.

8.4.2.5.1 Peak Hour

The results of the peak hour demand analysis by including the Buildout scenario did not differ significantly from the existing system analysis performed in Chapter 7. The results indicate that the future system in Pressure Zone 4 is adequate to deliver peak hour demands under the City's minimum pressure criteria of 40 psi (see Figure 8-17). Head losses in Pressure Zone 4 meet the City's maximum design criteria of 10 ft/kft.

8.4.2.5.2 Maximum Day plus Fire Flow Demand

Fire flows were simulated at hydrant locations throughout Zone 4 based on land use type. Results indicate that the future system in Pressure Zone 4 is able to meet the required residual pressure of 20 psi under maximum day plus fire flow demand conditions (see Figure 8-18).

8.5 SUMMARY OF RECOMMENDED IMPROVEMENTS FOR EXISTING POTABLE WATER SYSTEM

The recommended improvements needed to eliminate deficiencies identified in the analysis of the future distribution systems are summarized below.

8.5.1 Pipelines

The following improvements are recommended:

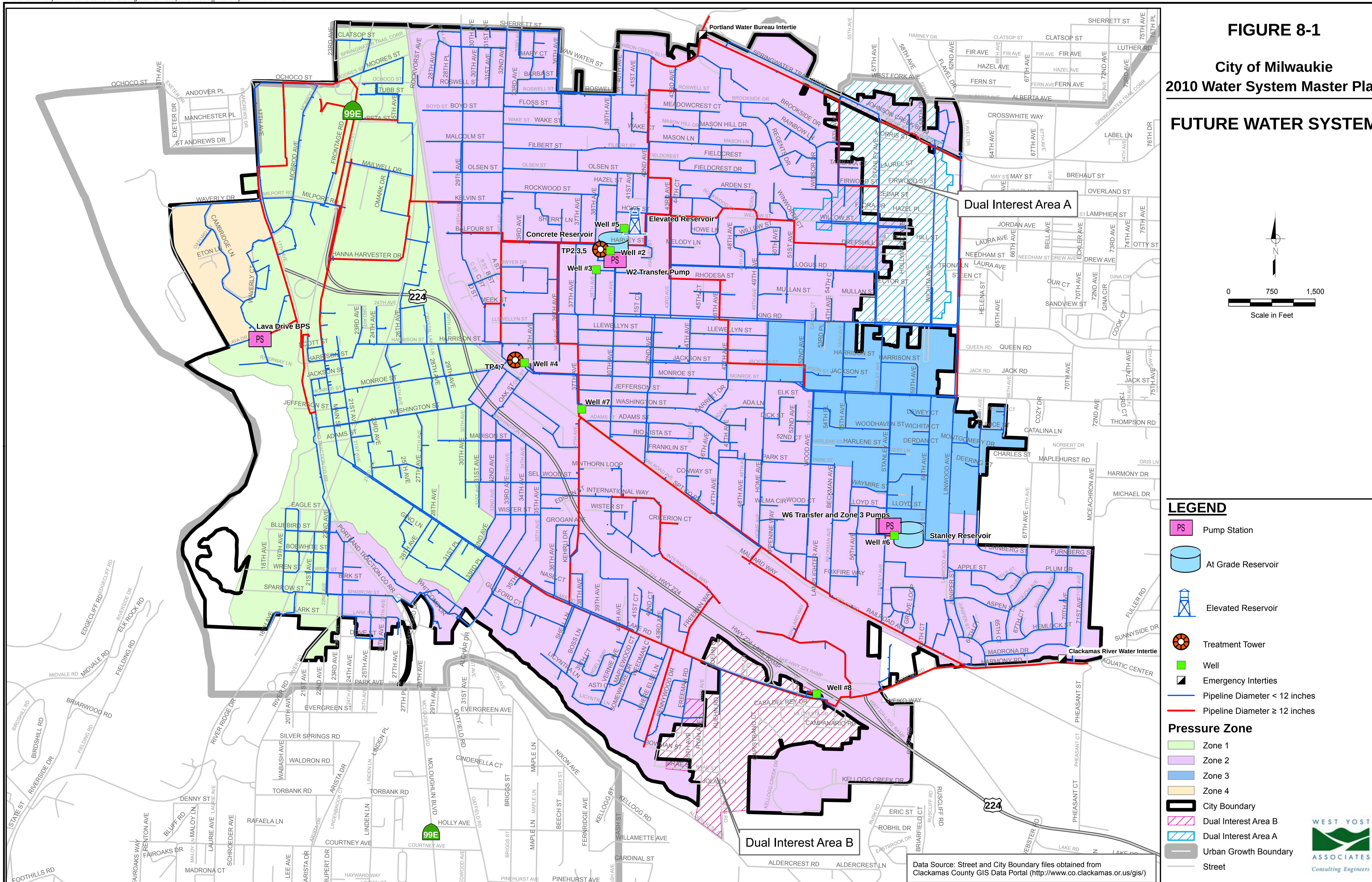
Dual Interest Area A and B Connection Projects

- Installation of approximately 6,060 linear feet of 8-inch diameter DI pipeline in Dual Interest Area A.
- Installation of approximately 4,570 linear feet of 8-inch diameter DI pipeline in Dual Interest Area B.

General Fire Flow Improvements

- Improve fire flow capacity in the existing water system as part of future pipeline replacement projects.

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 8\FIG8-1_Future System.mxd 5/2/2011



O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 8\FIG8-2_Land Use.mxd 5/2/2011

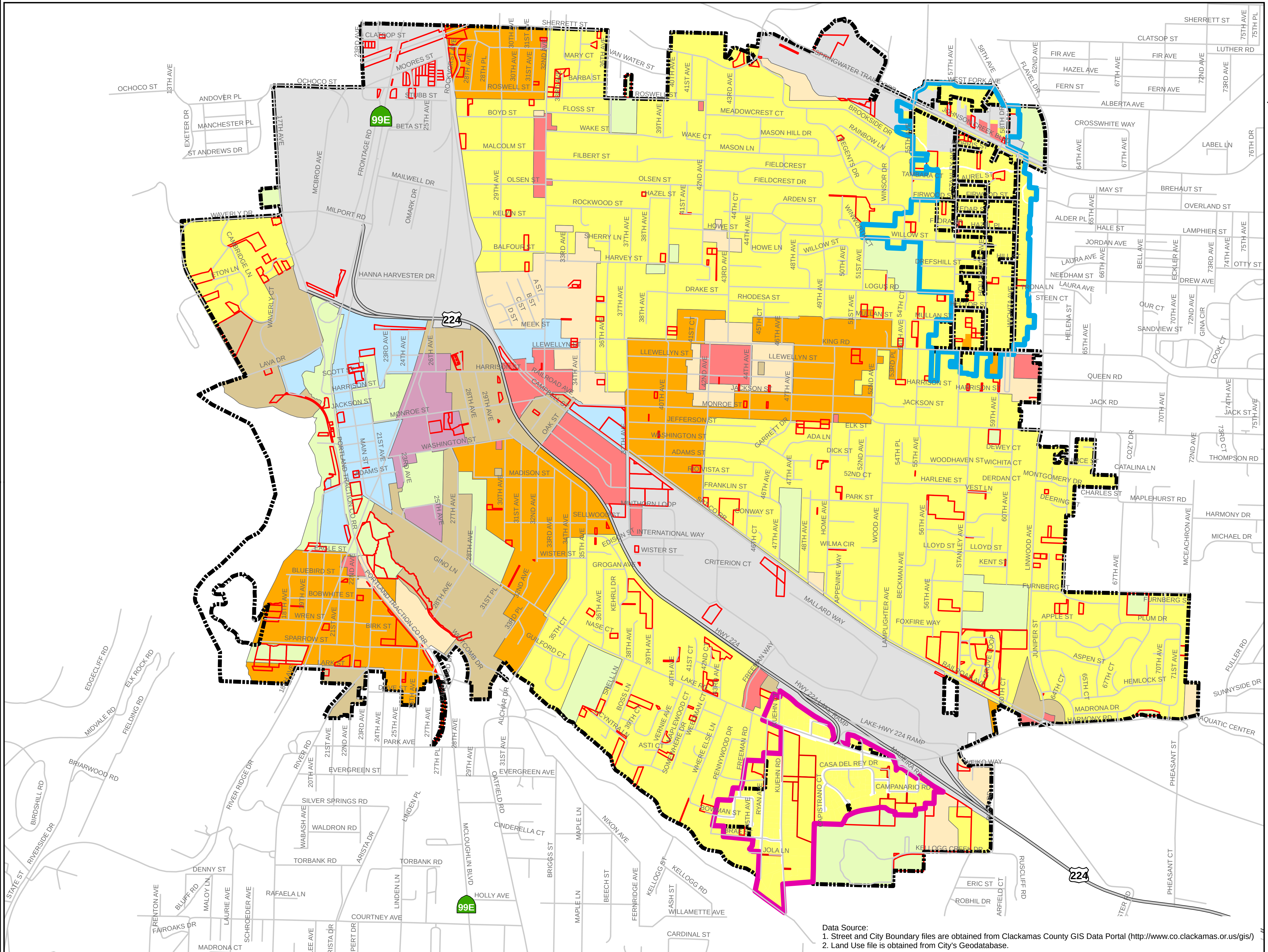


FIGURE 8-2

City of Milwaukie
2010 Water System Master Plan

GENERAL PLAN
LAND USE
VACANT PARCELS

N

0 750 1,500
Scale in Feet

LEGEND

- Low Density Residential
- Medium Density Residential
- Moderate Density Residential
- High Density Residential
- Mixed Use
- Commercial
- Industrial
- Public
- Town Center
- Vacant Parcel
- Dual Interest Area A
- Dual Interest Area B
- City Limit
- Street
- Highway or Freeway

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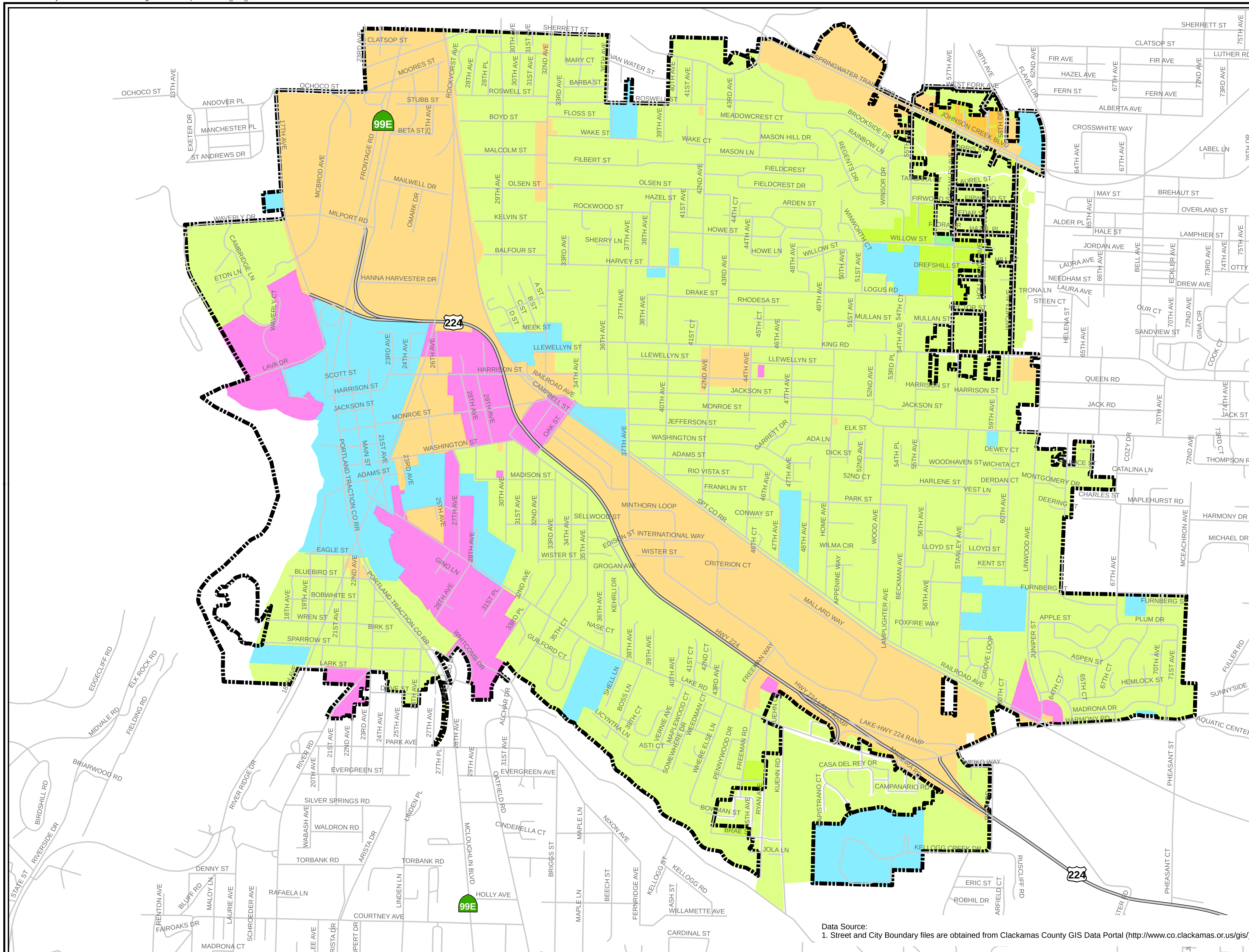
O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 8\FIG8-3_Fire_Flows.mxd 5/2/2011

FIGURE 8-3

City of Milwaukie 2010 Water System Master Plan

FUTURE SYSTEM FIRE FLOW DEMANDS



LEGEND

Fire Flow Demand

- 1,500 gpm for 2 hours
- 1,500 gpm for 3 hours
- 2,000 gpm for 4 hours
- 3,000 gpm for 4 hours
- 2010 City Limit
- Street
- Highway or Freeway

Data Source:
1. Street and City Boundary files are obtained from Clackamas County GIS Data Portal (http://www.co.clackamas.or.us/gis/)

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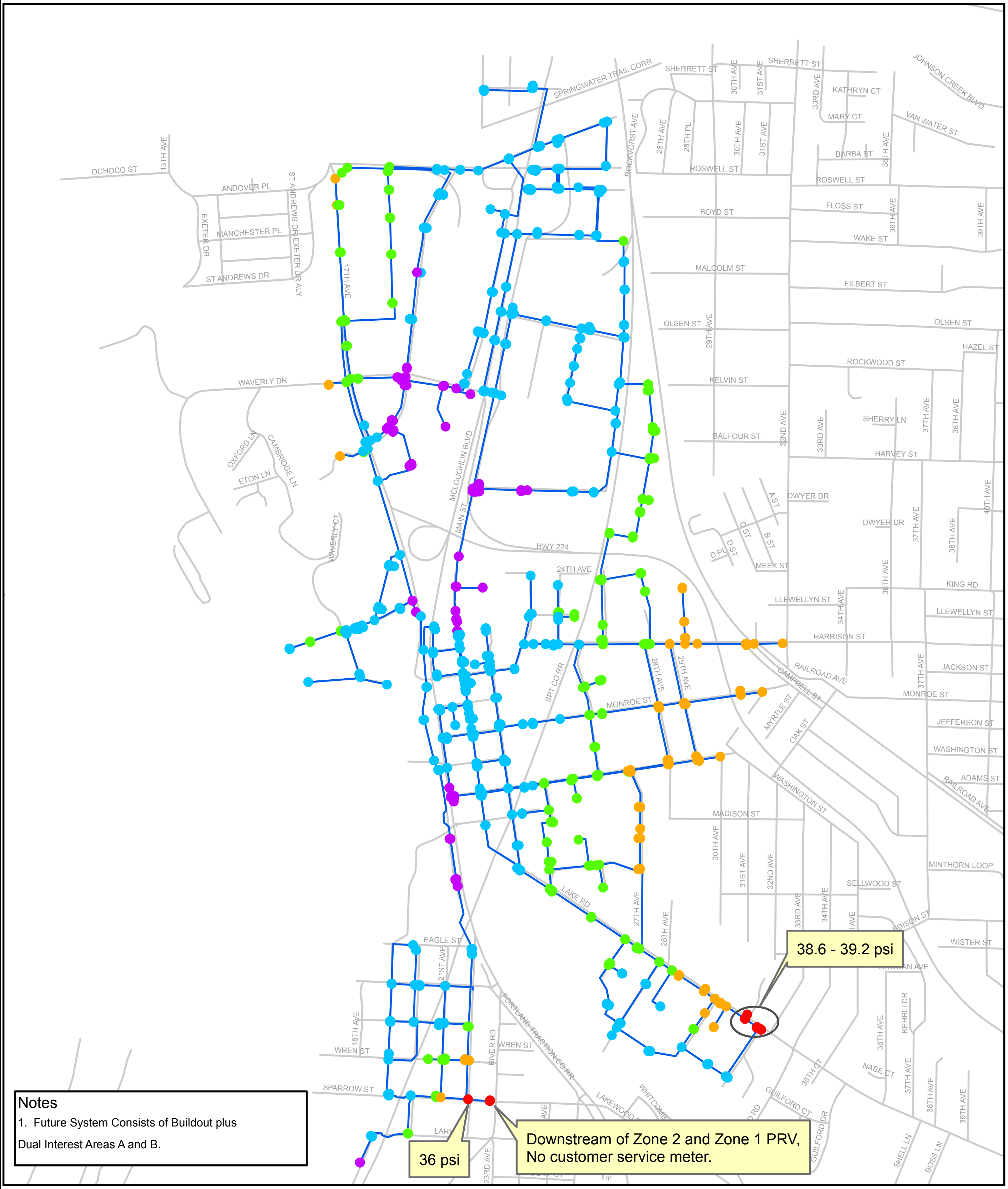

LEGEND

- Pressure < 40 psi
- 40 psi ≤ Pressure < 50 psi
- 50 psi ≤ Pressure < 60 psi
- 60 psi ≤ Pressure < 70 psi
- Pressure ≥ 70 psi
- HL/1000 < 10 ft/kft
- HL/1000 ≥ 10 ft/kft
- Street

N

0 650 1,300

Scale in Feet

FIGURE 8-4

City of Milwaukie
2010 Water System Master Plan
PRESSURE ZONE 1
PEAK HOUR ANALYSIS

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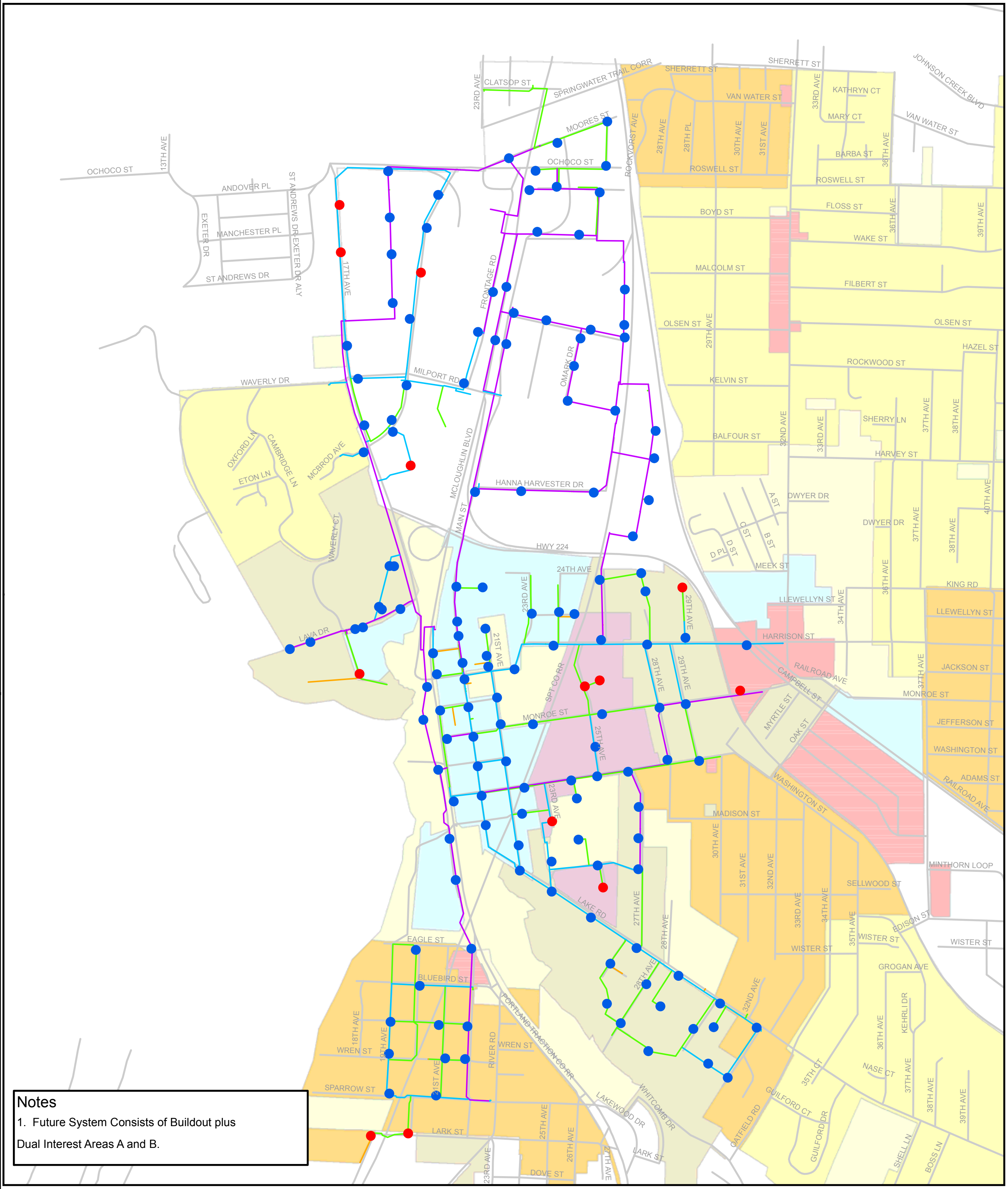
O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 8\FIG8-5_MDFF_Z1.mxd 8/23/2011



LEGEND

- Residual Pressure < 20 psi
- Residual Pressure ≥ 20 psi
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches
- Low Density Residential
- Medium Density Residential
- Moderate Density Residential
- High Density Residential
- Mixed Use
- Commercial
- Industrial
- Public
- Town Center
- Street

0 650 1,300
Scale in Feet

FIGURE 8-5

City of Milwaukie

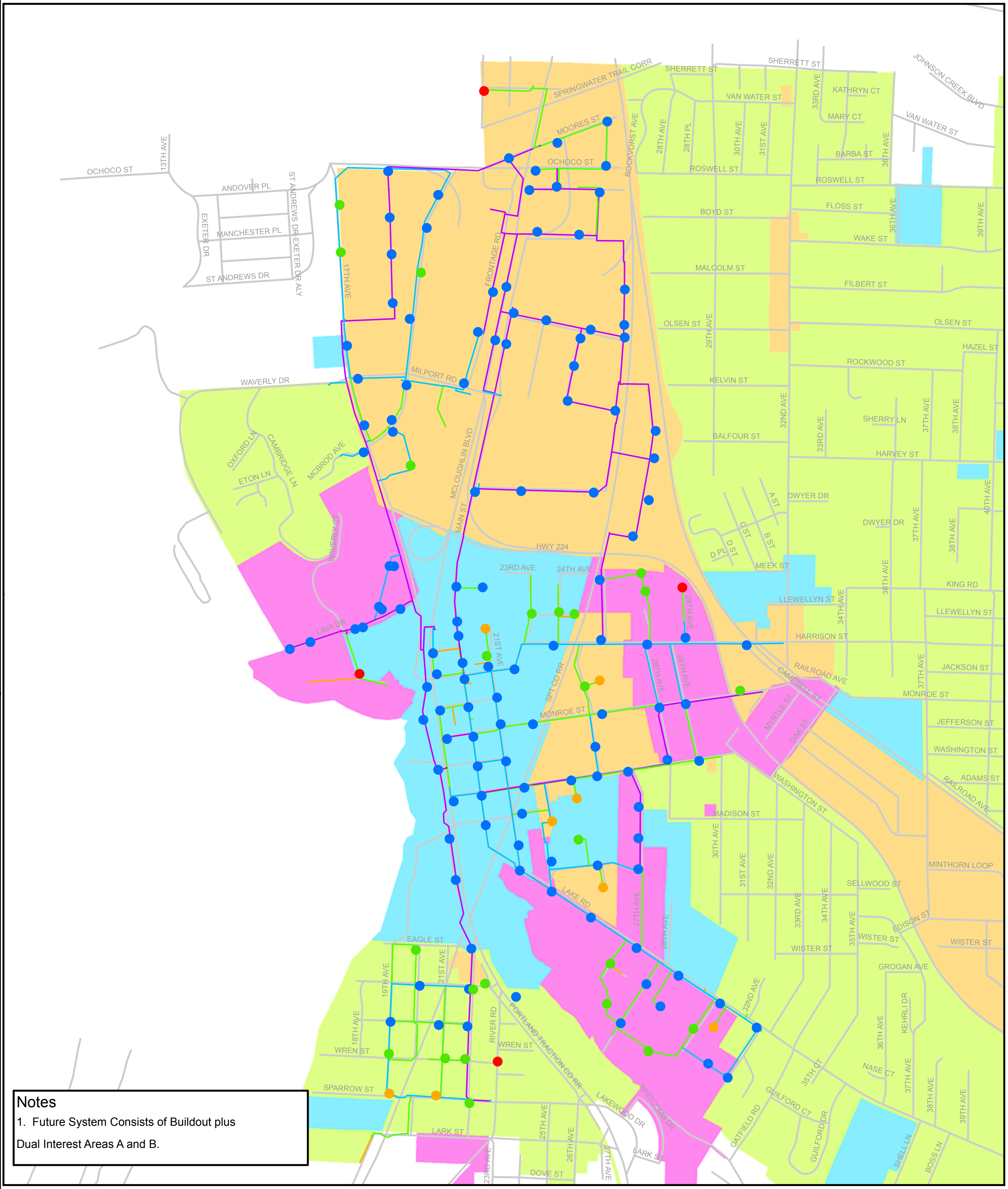
2010 Water System Master Plan

PRESSURE ZONE 1

MAX DAY PLUS FIRE FLOW

WEST YOST ASSOCIATES Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 8\FIG8-6_AvailFF_Z1.mxd



Notes

1. Future System Consists of Buildout plus Dual Interest Areas A and B.

LEGEND

- Fire Flow < 1,500 gom
- 1,500 gpm ≤ Fire Flow < 2,500 gpm
- 2,500 gpm ≤ Fire Flow < 3,000 gpm
- Fire Flow ≥ 3,000 gpm
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches

Fire Flow Demand

- 1,500 gpm for 2 hours
- 1,500 gpm for 3 hours
- 2,000 gpm for 4 hours
- 3,000 gpm for 4 hours
- Street

N

0 650 1,300

Scale in Feet

FIGURE 8-6

City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 1 AVAILABLE FIRE FLOW

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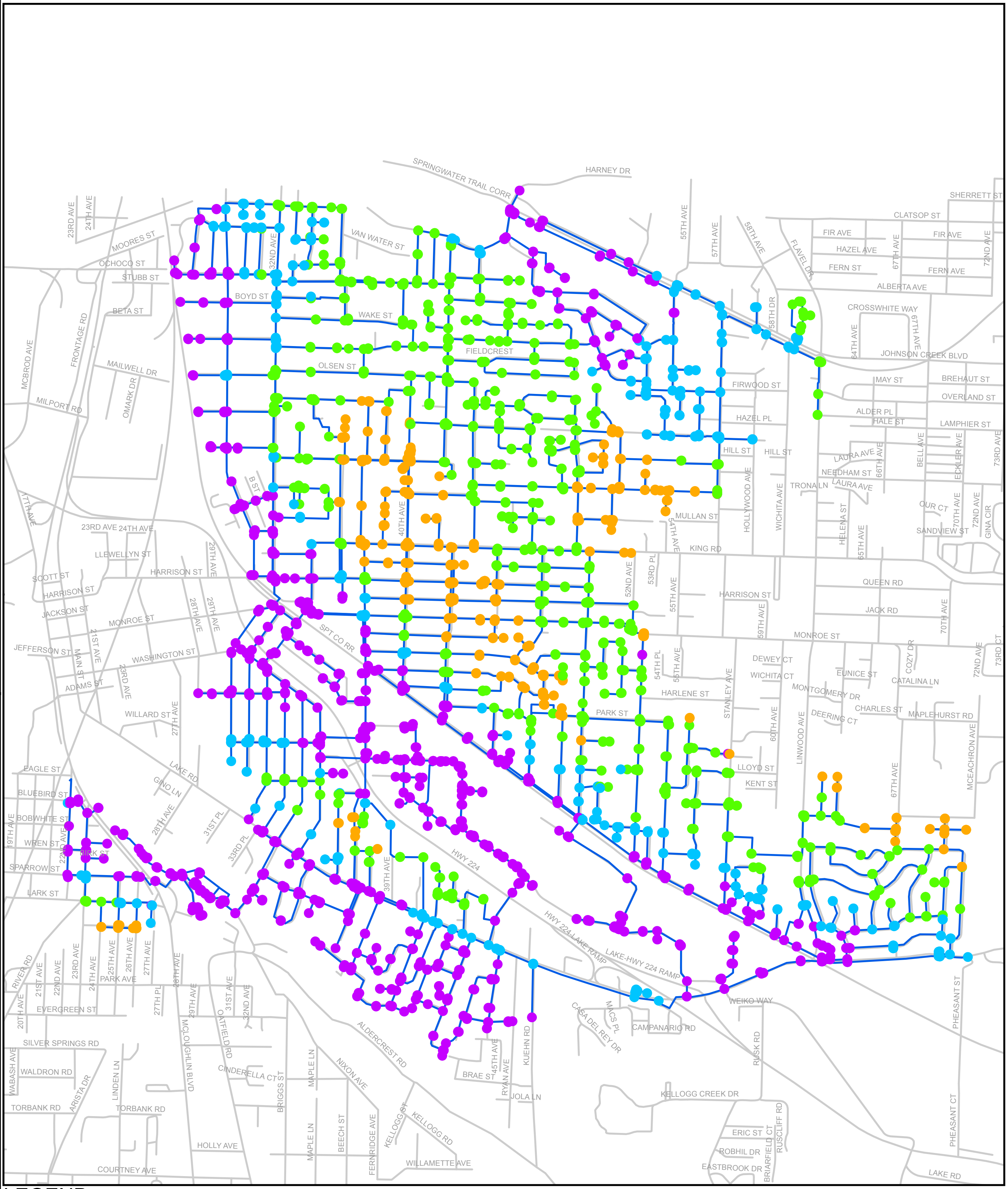
O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 8\FIG8-7_PH_Z2.mxd 8/23/2011



LEGEND

- 40 psi ≤ Pressure < 50 psi
- 50 psi ≤ Pressure < 60 psi
- 60 psi ≤ Pressure < 70 psi
- Pressure ≥ 70 psi
- HL/1000 < 10 ft/kft
- HL/1000 ≥ 10 ft/kft
- Street

N

0 1,000 2,000

Scale in Feet

FIGURE 8-7

City of Milwaukie
2010 Water System Master Plan
PRESSURE ZONE 2
PEAK HOUR ANALYSIS
BUILDOUT SYSTEM

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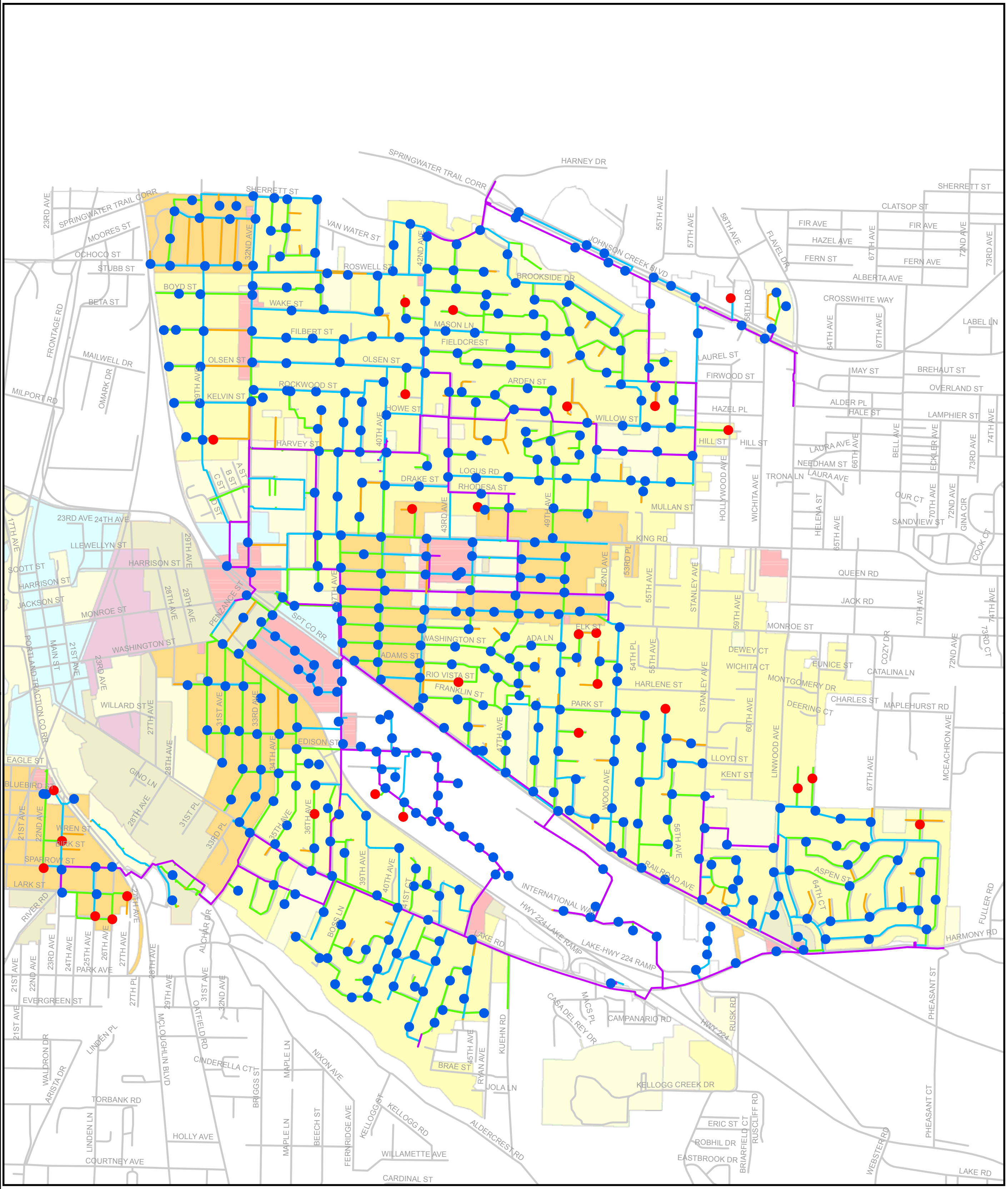

LEGEND

- Residual Pressure < 20 psi
- Residual Pressure ≥ 20 psi
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches
- Low Density Residential
- Medium Density Residential
- Moderate Density Residential
- High Density Residential
- Mixed Use
- Commercial
- Industrial
- Public
- Town Center
- Street

N

0 1,000 2,000

Scale in Feet

FIGURE 8-8

City of Milwaukie
2010 Water System Master Plan
PRESSURE ZONE 2
MAX DAY PLUS FIRE FLOW
BUILDOUT SYSTEM

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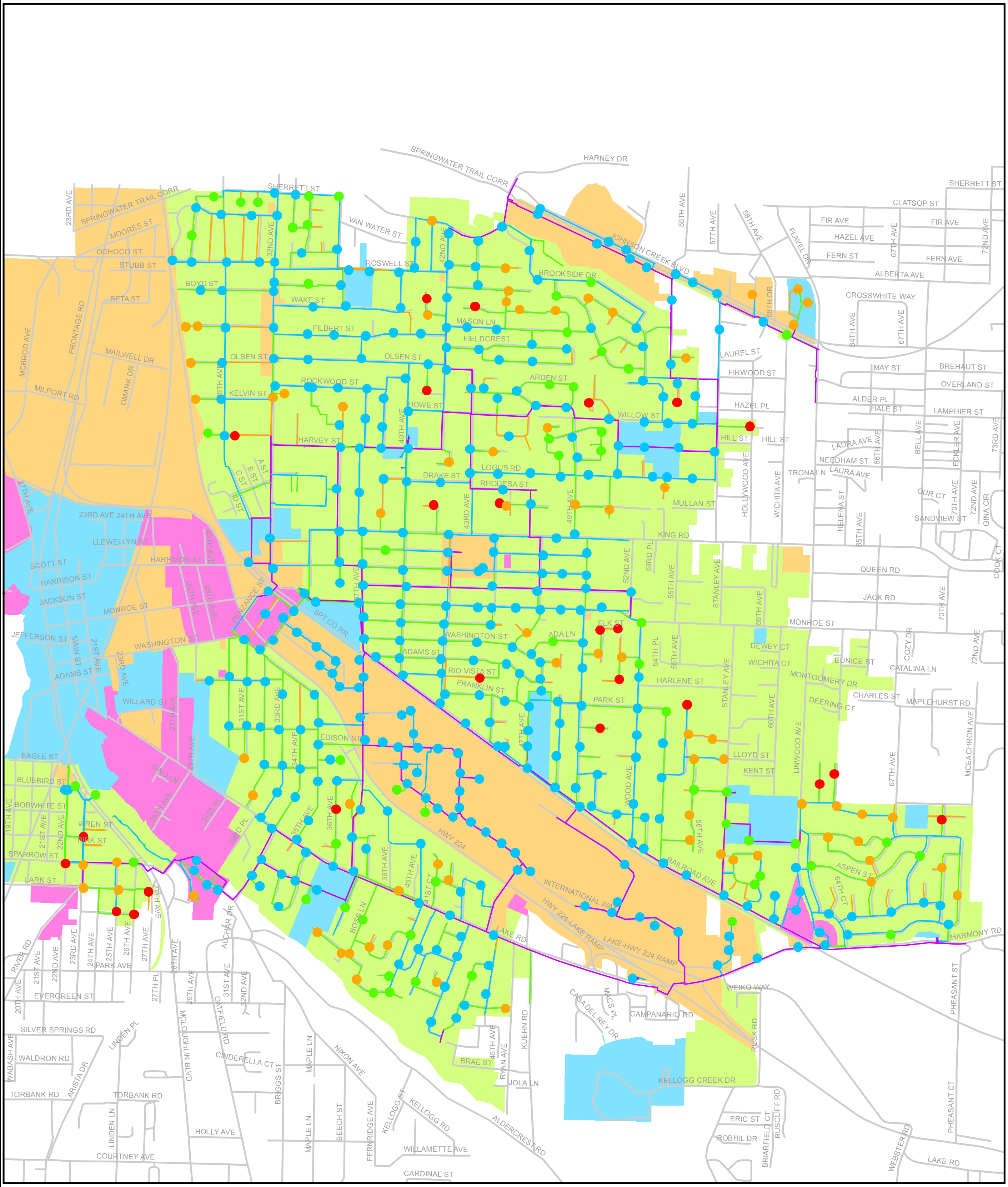
O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 8\FIG8-9_AvailFF_Z2.mxd



LEGEND

- Fire Flow < 1,500 gpm
- 1,500 pgm ≤ Fire Flow < 2,500 gpm
- 2,500 pgm ≤ Fire Flow < 3,000 gpm
- Fire Flow ≥ 3,000 gpm
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches

Fire Flow Demand

- 1,500 gpm for 2 hours
- 1,500 gpm for 3 hours
- 2,000 gpm for 4 hours
- 3,000 gpm for 4 hours
- Street

N

0 1,000 2,000

Scale in Feet

FIGURE 8-9

City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 2 AVAILABLE FIRE FLOW BUILDOUT SYSTEM

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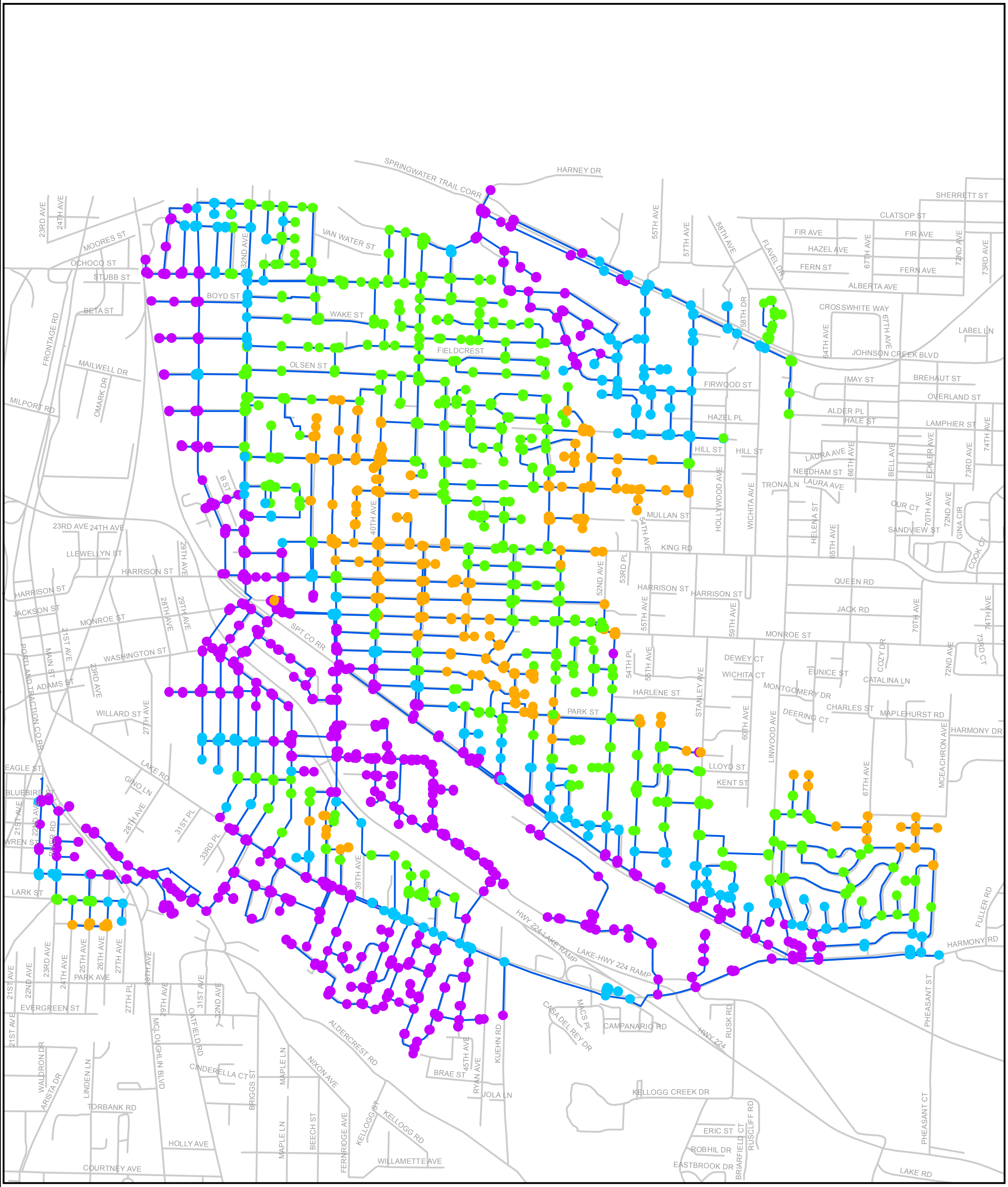

LEGEND

- 40 psi ≤ Pressure < 50 psi
- 50 psi ≤ Pressure < 60 psi
- 60 psi ≤ Pressure < 70 psi
- Pressure ≥ 70 psi
- HL/1000 < 10 ft/kft
- HL/1000 ≥ 10 ft/kft
- Street

N

0 1,000 2,000

Scale in Feet

FIGURE 8-10

City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 2

PEAK HOUR ANALYSIS

BUILDOUT + DUAL INTEREST AREAS

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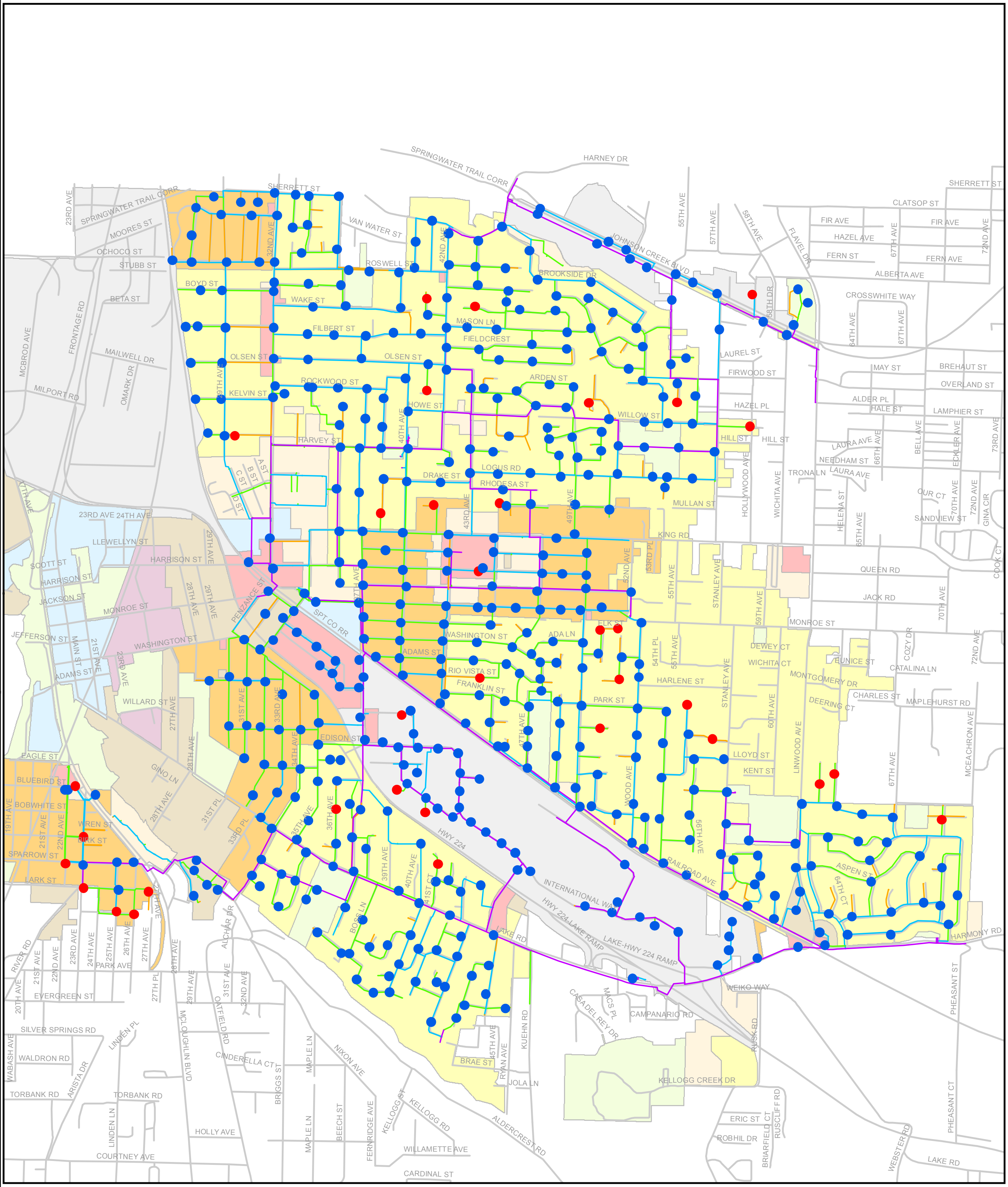
O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 8\FIG8-11_MDFFBOAB_Z2.mxd



LEGEND

- Residual Pressure < 20 psi
- Residual Pressure ≥ 20 psi
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches
- Low Density Residential
- Medium Density Residential
- Moderate Density Residential
- High Density Residential
- Mixed Use
- Commercial
- Industrial
- Public
- Town Center
- Street

N

0 1,000 2,000

Scale in Feet

FIGURE 8-11

City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 2

MAX DAY PLUS FIRE FLOW

BUILDOUT + DUAL INTEREST AREAS

WEST YOST ASSOCIATES

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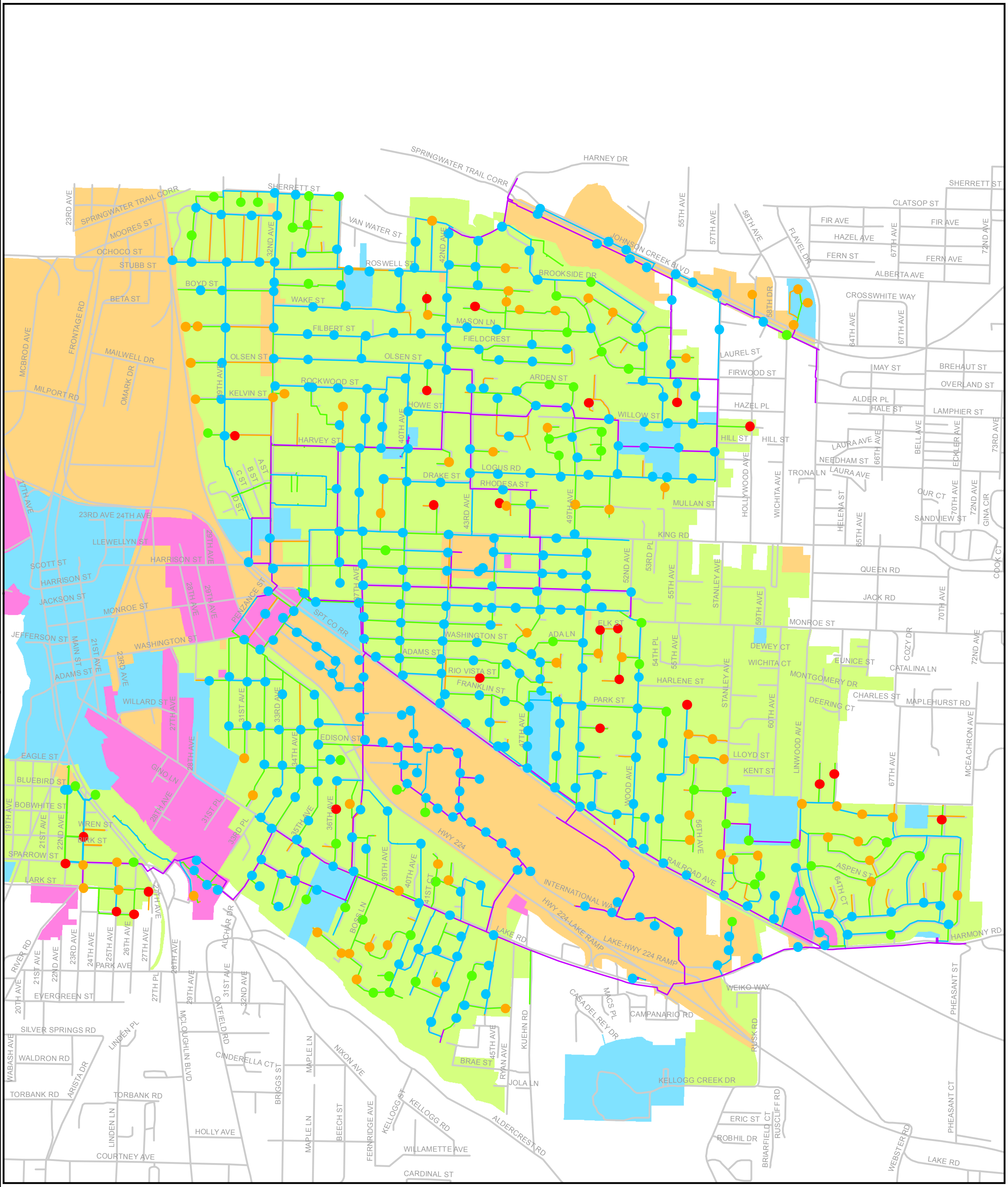
O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 8\FIG8-12_AvailFFBOAB_Z2.mxd



LEGEND

- Fire Flow < 1,500 gpm
- 1,500 gpm ≤ Fire Flow < 2,500 gpm
- 2,500 gpm ≤ Fire Flow < 3,000 gpm
- Fireflow ≥ 3,000 gpm
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches

Fire Flow Demand

- 1,500 gpm for 2 hours
- 1,500 gpm for 3 hours
- 2,000 gpm for 4 hours
- 3,000 gpm for 4 hours
- Street

N

0 1,000 2,000

Scale in Feet

FIGURE 8-12

City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 2
AVAILABLE FIRE FLOW
BUILDOUT + DUAL INTEREST AREAS

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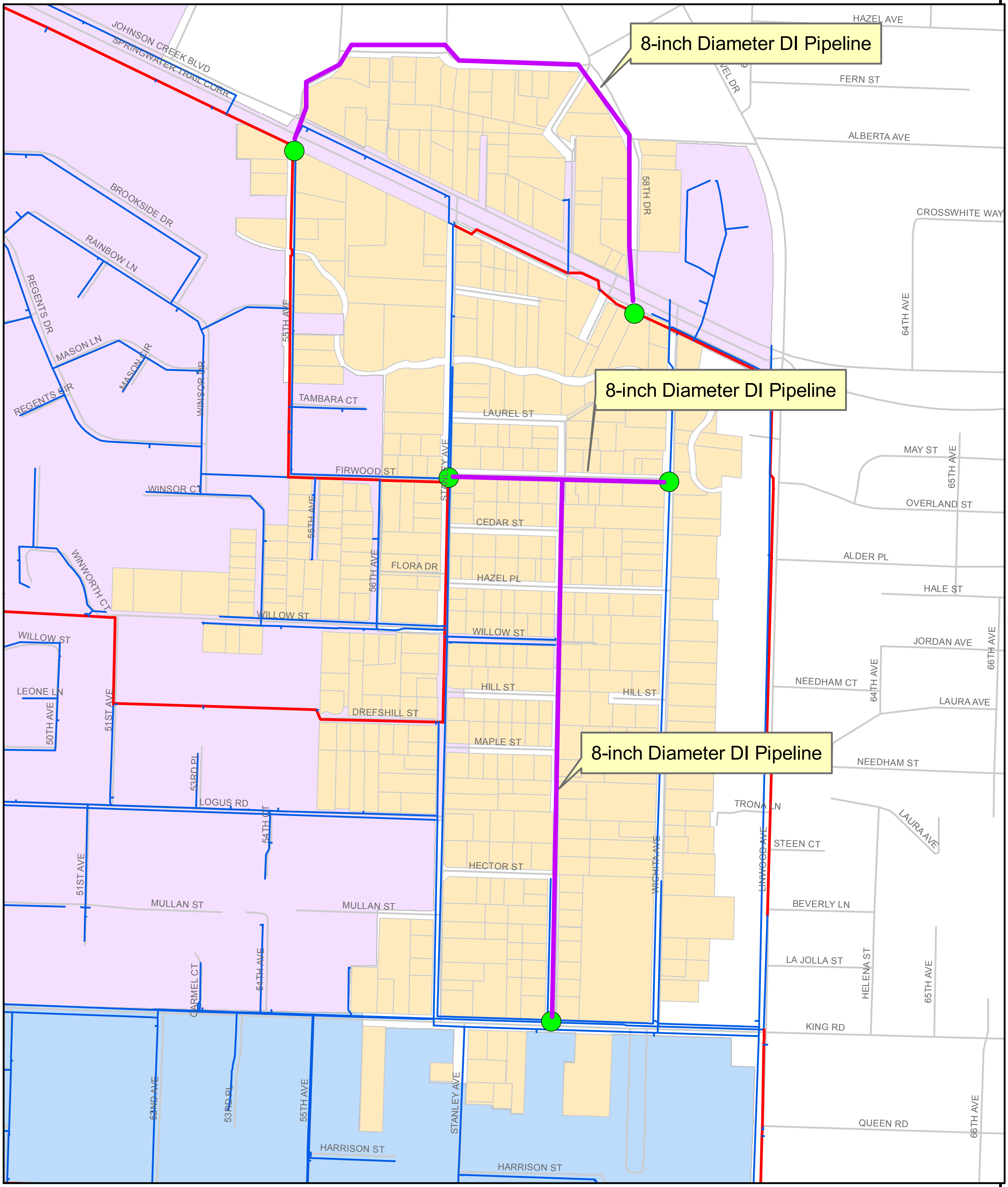
O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 8\FIG8-13_Area_A_Improvements.mxd



LEGEND

- Connection to Existing System
- Existing Pipeline Diameter < 12 inches
- Existing Pipeline Diameter ≥ 12 inches
- Dual Interest Area A
- Zone 1
- Zone 2
- Zone 3
- Zone 4

N

0 300 600

Scale in Feet

FIGURE 8-13

City of Milwaukie

2010 Water System Master Plan

DUAL INTEREST AREA A

RECOMMENDED IMPROVEMENTS

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O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 8\FIG8-14_Area_B_Improvements.mxd

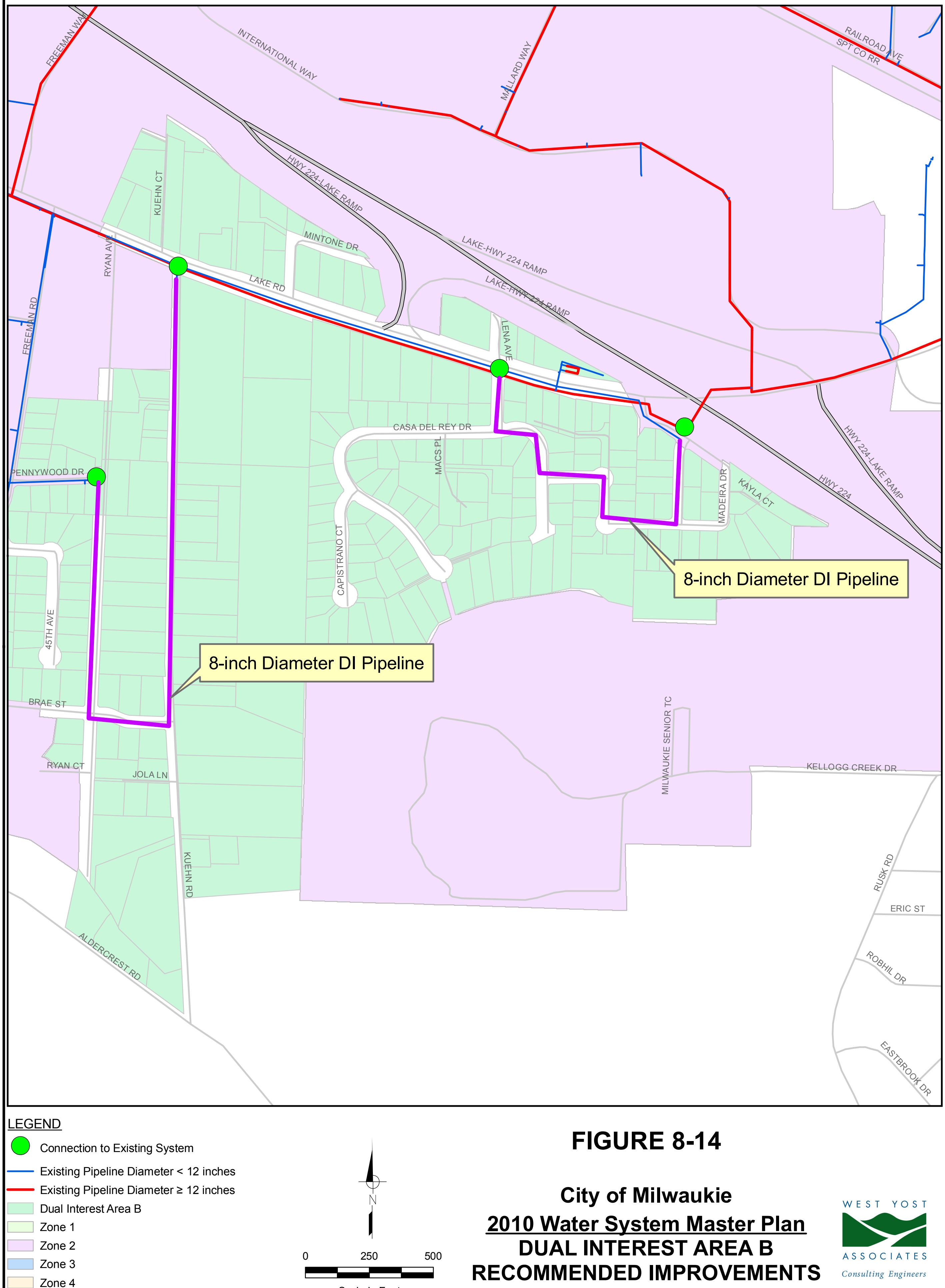

FIGURE 8-14

City of Milwaukie
2010 Water System Master Plan
DUAL INTEREST AREA B
RECOMMENDED IMPROVEMENTS

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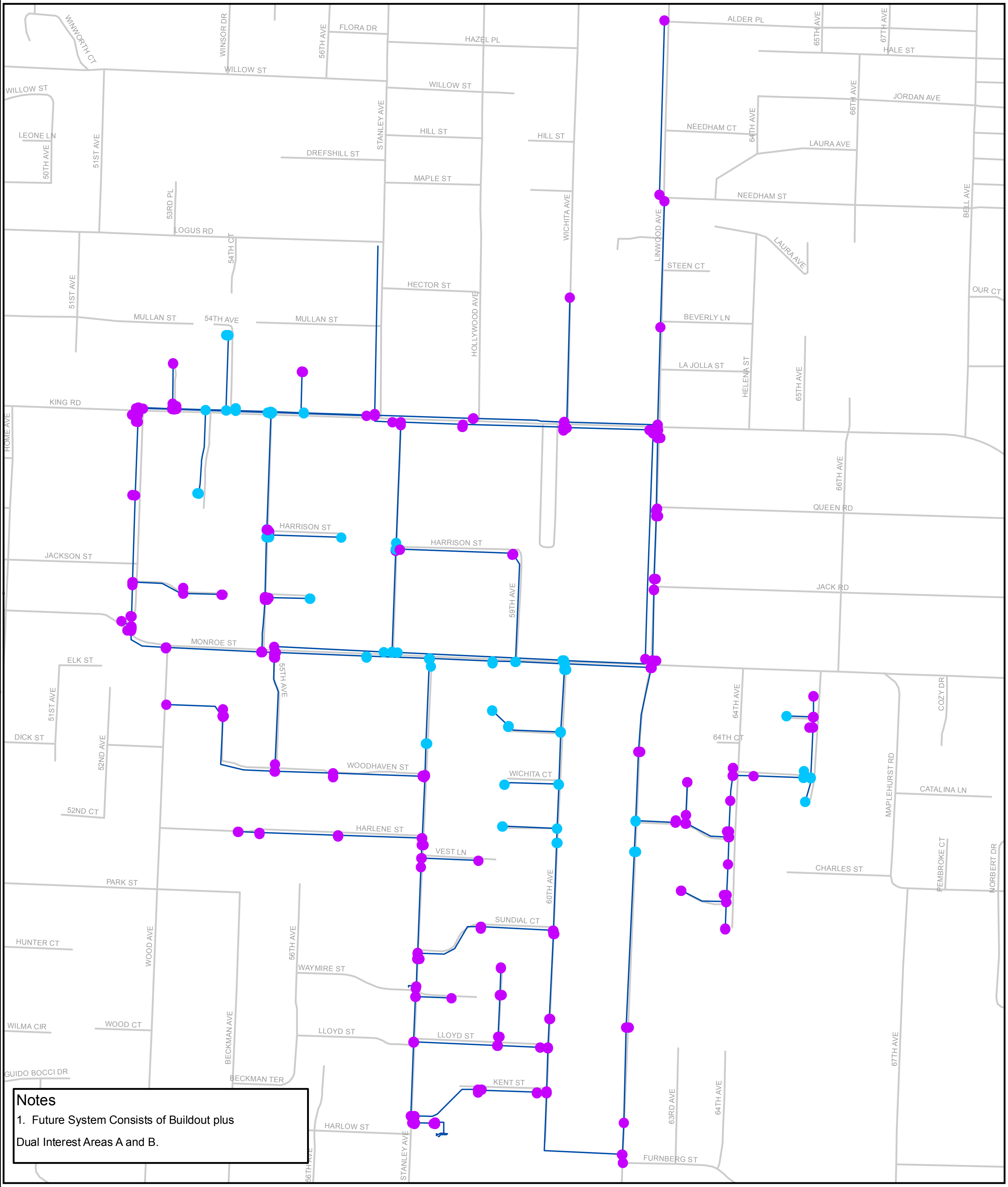

LEGEND

- 60 psi ≤ Pressure < 70 psi
- Pressure ≥ 70 psi
- HL/1000 < 10 ft/kft
- Street

N

0 350 700

Scale in Feet

FIGURE 8-15

City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 3
PEAK HOUR ANALYSIS

WEST YOST ASSOCIATES

Consulting Engineers

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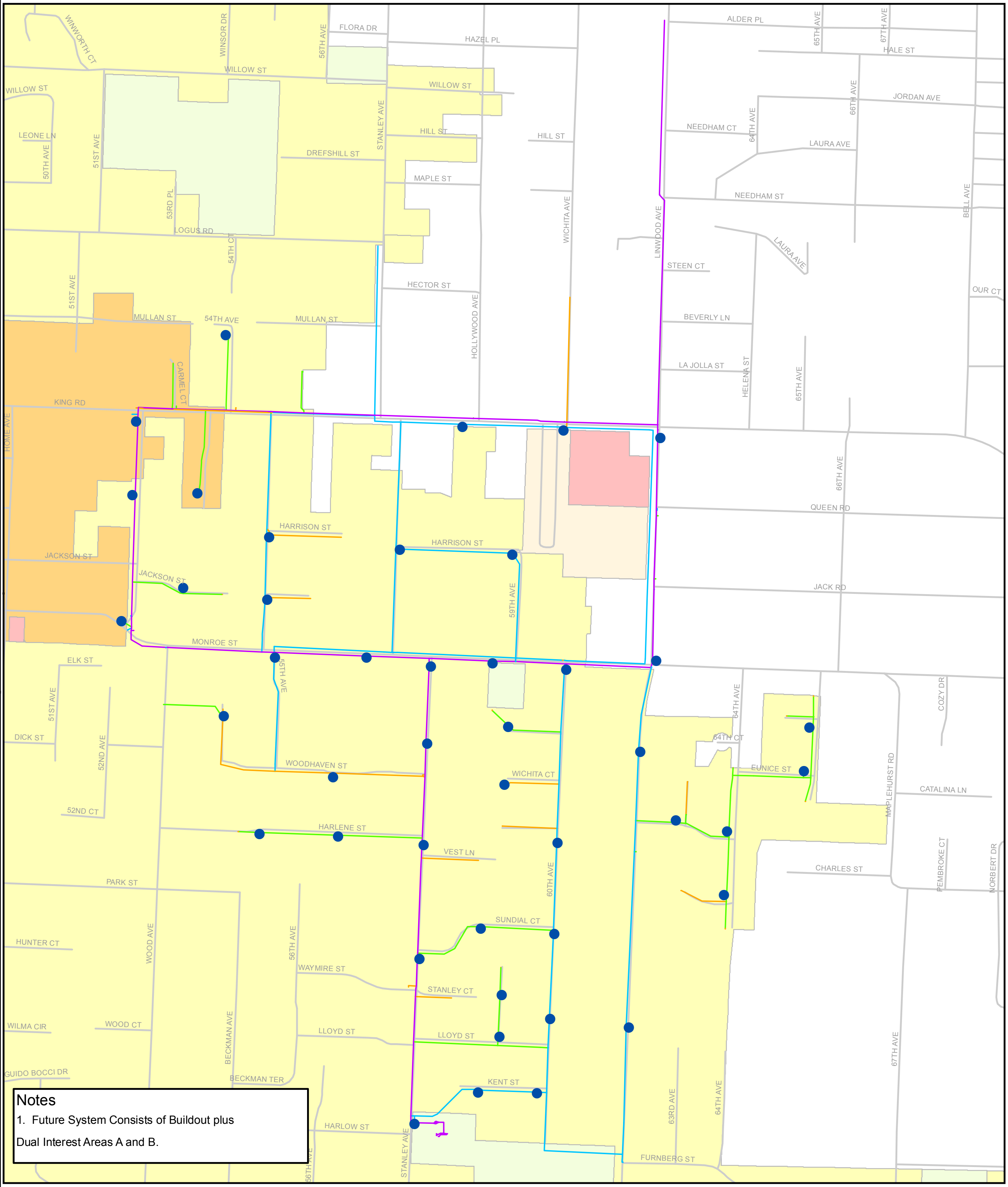

LEGEND

- Residual Pressure ≥ 20 psi
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches
- Low Density Residential
- Medium Density Residential
- Moderate Density Residential
- High Density Residential
- Mixed Use
- Commercial
- Industrial
- Public
- Town Center
- Street

N

0 350 700

Scale in Feet

FIGURE 8-16

City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 3

MAX DAY PLUS FIRE FLOW

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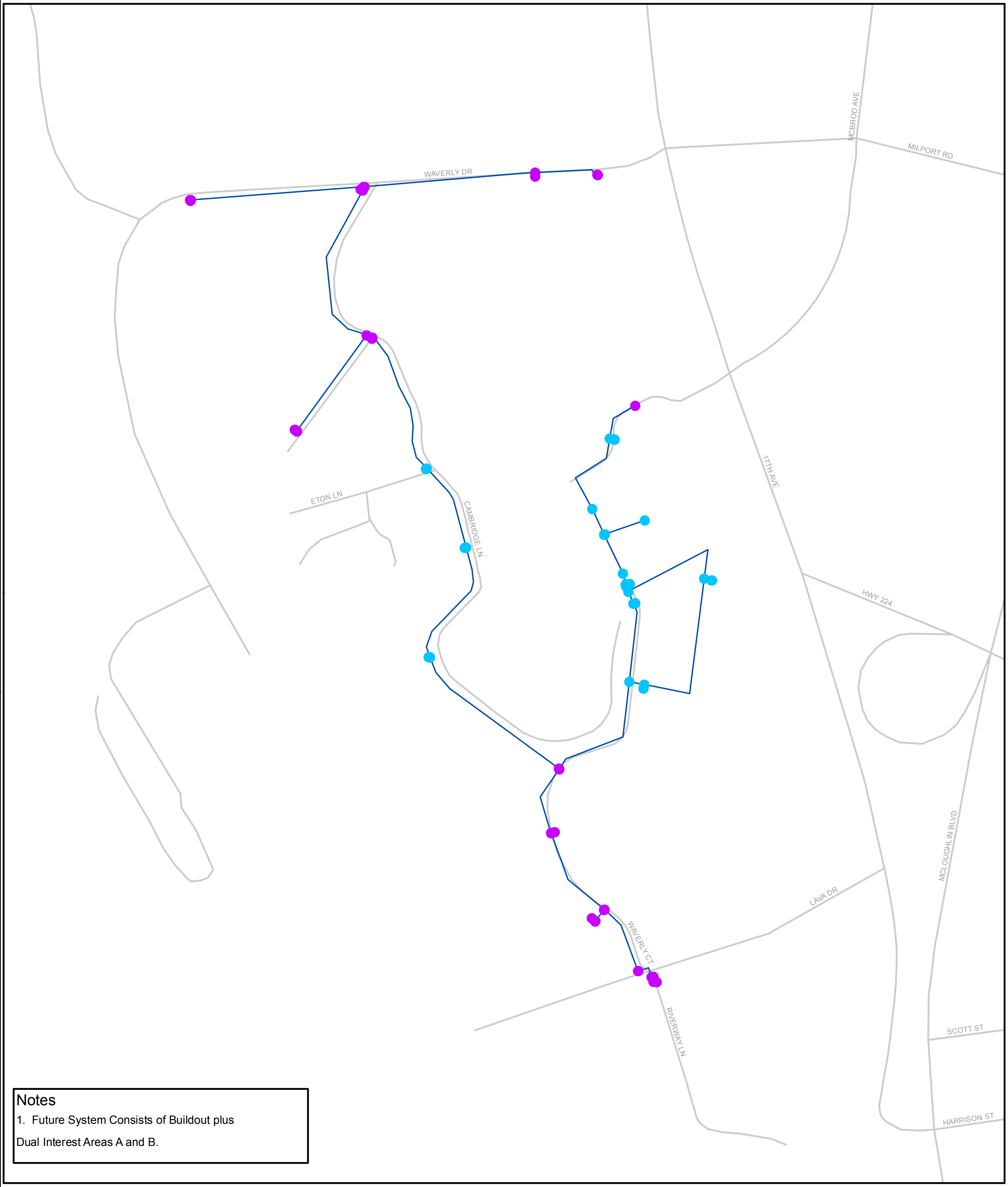

Notes

1. Future System Consists of Buildout plus Dual Interest Areas A and B.

LEGEND

- 60 psi ≤ Pressure < 70 psi
- Pressure ≥ 70 psi
- HL/1000 < 10 ft/kft
- HL/1000 ≥ 10 ft/kft
- Street

N

0 200 400

Scale in Feet

FIGURE 8-17

City of Milwaukie

2010 Water System Master Plan

PRESSURE ZONE 4

PEAK HOUR ANALYSIS

WEST YOST ASSOCIATES

Consulting Engineers

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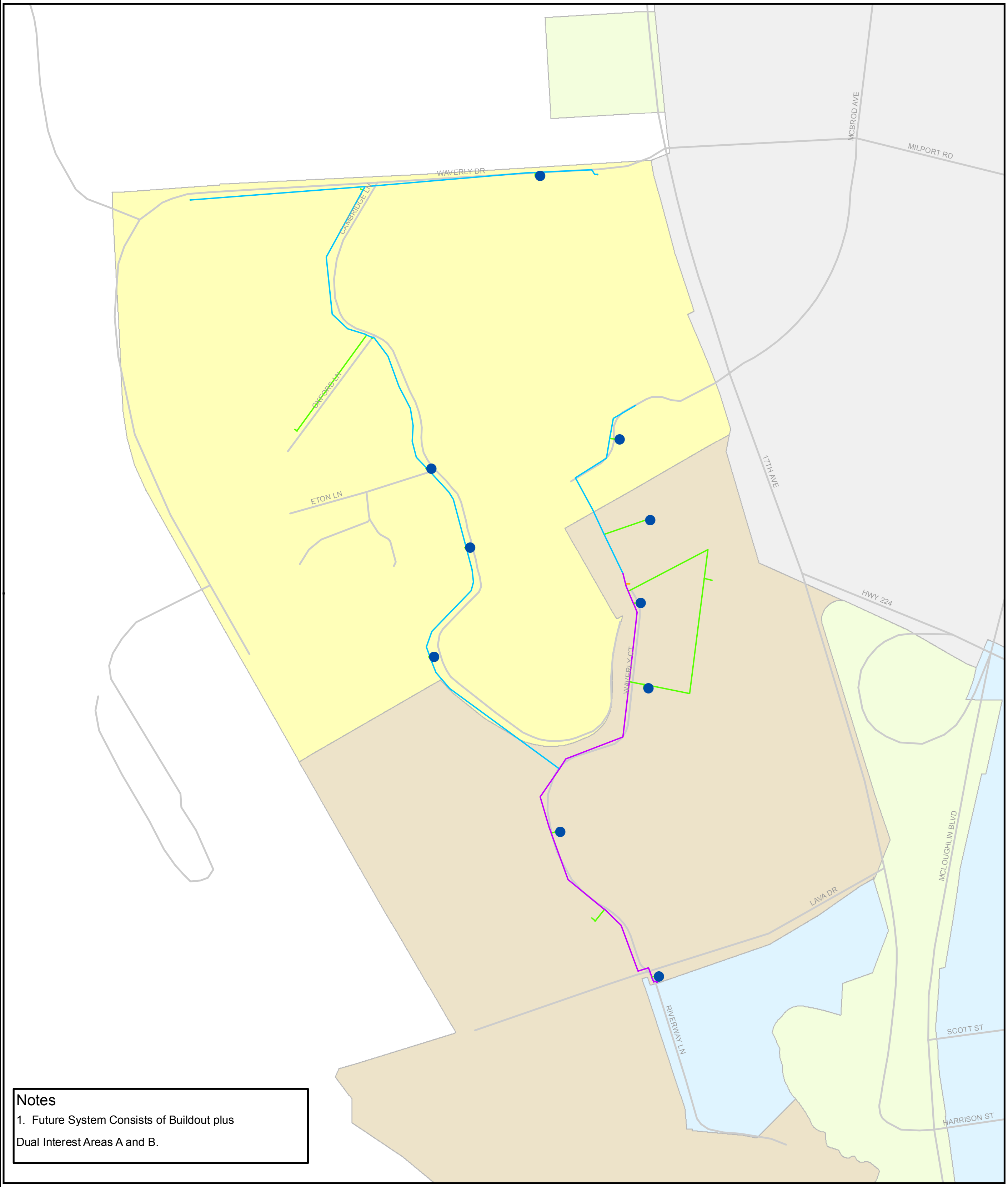

Notes
1. Future System Consists of Buildout plus Dual Interest Areas A and B.

LEGEND

- Residual Pressure > 20 psi
- Pipeline Diameter < 6 inches
- Pipeline Diameter = 6 inches
- Pipeline Diameter = 8 inches
- Pipeline Diameter > 8 inches
- Low Density Residential
- Medium Density Residential
- Moderate Density Residential
- High Density Residential
- Mixed Use
- Commercial
- Industrial
- Public
- Town Center
- Street

N

0 200 400
Scale in Feet

FIGURE 8-18

City of Milwaukie
2010 Water System Master Plan
PRESSURE ZONE 4
MAX DAY PLUS FIRE FLOW

WEST YOST ASSOCIATES
Consulting Engineers

CHAPTER 9
Recommended Capital Improvement Program

This chapter presents the recommended CIP for the City's existing and buildout water system. Several recommendations for improvements to the existing and buildout water system are described previously in Chapters 7 and 8, respectively. This chapter provides descriptions of the recommended CIP program, along with estimates of probable construction costs. The intended timeframe for the completion of the projects in the recommended CIP is 10 years.

The intended funding mechanism for the recommended CIP is water utility fees and System Development Charges (SDCs) collected as new development occurs. This is a "pay-as-you-go" funding approach. The City reviews capital improvement program funding through utility fee adjustments and SDC charges on an annual basis as part of the budgeting process. The specific timing of the projects identified in the recommended CIP is dependent upon the specific rate structure for water utility fees that is chosen by the City Council.

Costs are presented in January 2011 dollars based on an Engineering News Record Construction Cost Index (ENR CCI) of 8,938 (20 Cities Average). Total CIP costs include the following contingencies and project cost allowances:

- Construction Contingency: 20 percent
- Project Cost Allowances:
  - Design: 10 percent
  - Construction Management: 10 percent
  - Administration: 8 percent

A complete description of the assumptions used in developing the estimates of probable construction costs is provided in Appendix C.

9.1 EXISTING WATER SYSTEM IMPROVEMENTS

Chapter 7 provided an evaluation of the City's existing water system and its ability to meet the established operational and design criteria described in Chapter 5. Based on the evaluation, several improvements to the existing system were recommended to eliminate existing deficiencies, which are illustrated on Figure 9-1 and summarized as follows:

9.1.1 Water System Investigations and Studies

As part of the evaluation of the City's existing water system in Chapter 7, review of water system data and inspections, and interviews with City staff, there were a number of instances where additional information was needed to evaluate the condition of water system infrastructure. The following is a list of recommended water system investigations and studies to further determine the condition of water system infrastructure and to schedule future improvement projects as needed.

- Conduct a comprehensive structural and operational inspection of the Concrete Reservoir, including recommendations on needed improvements and expected remaining operational life of the current reservoir. This inspection is needed due to the age of the reservoir and the retrofitted liner installed in 1995.

- Conduct a comprehensive operational inspection to reconfigure a portion of the Southwest corner of Pressure Zone 1 so that it is served by Pressure Zone 2. This includes a verification of the connection between Kellogg Lake Apartments to SE River Road, and the isolation of the 6-inch diameter pipeline along SE 22nd Avenue from Zone 1.

9.1.2 Water Storage Improvements

- Install a remote controlled shut-off valve or seismic valve at the Elevated Reservoir.
- Install a remote controlled shut-off valve or seismic valve at the Concrete Reservoir.
- Install a remote controlled shut-off valve or seismic valve at the Stanley Reservoir.

9.1.3 Water Pumping Improvements

- Install two additional 1,750 gpm fire flow pumps to the Third Pressure Zone Booster Pump Station.

9.1.4 Water Pipeline Improvements

As shown in Chapter 7, there are system deficiencies for both peak hour and fire flow demand conditions. As discussed previously, the peak hour deficiencies were mitigated through the reconfiguration of a portion of Pressure Zone 1 into Pressure Zone 2. However, the fire flow deficiencies will require a large number of projects to eliminate these deficiencies. A series of queries have been conducted to identify those pipeline replacements that are most urgent. These improvement projects have been prioritized based on the existing water system's size, age, and proximity to public facilities. The highest priority projects are included in the CIP and are prioritized as follows:

- Peak hour deficiencies.
- Small diameter steel pipe: None Identified.
- 2-inch diameter pipes that are part of a looped system: None Identified.
- Fire flow deficiencies in areas zoned "public".
- 4-inch diameter pipe installed before 1960.
- 6-inch diameter pipe installed before 1960.

In addition, pipelines slated for replacement should be moved ahead on the priority list if a street is scheduled for resurfacing. A summary of water pipeline improvement projects as determined by the criteria above is presented in Table 9-1.

Table 9-1. Recommended Pipeline CIP for Existing System

CIP ID	Pressure Zone	Description of Location	Length, feet	Diameter, inches	
				Existing	Recommended
PH01	1	Reconfigure Southwest portion of Zone 1 Boundary	450	-	8
Public Area Fire Flow Improvements					
FF01	1	From hydrant to Willard Street	320	6	8
FF02	2	Area west of Flavel Drive	600	6	8
Fire Flow Improvements: Pipelines Constructed Prior to 1960					
FF03	1	See Figure 9-1	10	4	8
FF03	2	See Figure 9-1	10,582	4	8
FF03	3	See Figure 9-1	2,975	4	8
FF03	1	See Figure 9-1	15,156	6	8
FF03	2	See Figure 9-1	49,373	6	8
FF03	3	See Figure 9-1	5,329	6	8
FF03	4	See Figure 9-1	361	6	8

PH : Indicates a project to resolve peak hour deficiencies
FF: indicates a project to resolve fire flow deficiencies.

9.1.5 Water System Facility Maintenance

As part of the evaluation of the City's existing water system in Chapter 7, review of water system data and inspections, and interviews with City staff, a number of facility maintenance projects were determined. These projects do not add capacity or reduce demand on the water system, but are meant to extend the remaining useful life of water system facilities. The recommended projects include the following:

- Prepare and recoat the exterior of the Stanley Tank.
- Prepare and recoat the top of the exterior of the Elevated Tank.
- Perform periodic well maintenance, including well pump removal and rehabilitation.

9.2 BUILDOUT WATER SYSTEM IMPROVEMENTS

Chapter 8 provided an evaluation of the City's buildout water system and its ability to meet the established operational and design criteria described in Chapter 5. Based on the evaluation, several improvements were recommended for the City's water system to meet buildout demands and are illustrated on Figure 9-2. These include the following:

9.2.1 Water Storage Improvements

The existing water storage, with the improvements identified in Section 9.1 in place, is able to meet the projected increase in water demands under the Buildout and Dual Interest A and B scenarios. No water storage improvements have been identified.

9.2.2 Water Pumping Improvements

The existing water pumping, with the improvements identified in Section 9.1 in place, is able to meet the projected increase in water demands under the Buildout and Dual Interest Area A and B scenarios. No water pumping improvements have been identified.

9.2.3 Water Pipeline Improvements

As discussed in Section 9.1.4, significant existing system deficiencies were identified under the maximum day plus fire flow demand condition. As a result, improvement projects have been prioritized based on the existing water system's size, age, and proximity to public facilities. Because the existing deficiencies affect the entire water pipeline system, the project prioritization also applies to the Buildout and Dual Interest Area A and B scenarios. The highest priority projects are included in the CIP and are prioritized as follows:

- Small diameter steel pipe: None Identified.
- 2-inch diameter pipes that are part of a looped system: None Identified.
- Installation of approximately 6,060 linear feet of 8-inch diameter DI pipeline to support the annexation of Dual Interest Area A.
- Installation of approximately 4,570 linear feet of 8-inch diameter DI pipeline to support the annexation of Dual Interest Area B.

In addition, pipelines slated for replacement should be moved ahead on the priority list if a street is scheduled for resurfacing. A summary of water pipeline improvement projects as determined by the criteria above are summarized in Table 9-2.

Table 9-2. Recommended Pipelines CIP for Buildout System

CIP ID	Pressure Zone	Description	Length, feet	Diameter, inches	
				Existing	Recommended
BDIA01	2	Infrastructure to support Dual Interest Area A	6,060	NA	8
BDIA02	2	Infrastructure to support for Dual Interest Area B	4,570	NA	8

9.2.4 Water Storage Improvements

As part of the evaluation of the City's buildout water system in Chapter 8, review of water system data and inspections, and interviews with City staff, a number of facility maintenance projects were determined. These projects do not add capacity or reduce demand on the water system, but are meant to extend the remaining useful life of water system facilities. The recommended projects include the following:

- Perform periodic well maintenance, including well pump removal and rehabilitation.

9.3 RECOMMENDED CIP COSTS

The recommended existing system CIP projects are presented in Table 9-3 along with their probable construction costs. The buildout system CIP projects are presented in Table 9-4, along with their probable construction costs. As shown, the existing system CIP costs are estimated to be $20.54 million. The buildout system CIP costs are estimated to be $2.64 million.

Table 9-3. Summary of Probable Construction Costs for Existing System CIP[(a)]					
Improvement Type	Improvement Description	CIP ID	Quantity	Estimated Construction Cost	CIP Cost (including contingency and cost allowances)
EXISTING CAPITAL IMPROVEMENTS					
Pipeline Improvement	Upsize hydrant pipeline in Zone 1	FF01	320 lf	$ 47,138	$ 72,403
Pipeline Improvement	Upsize hydrant pipeline in Zone 2	FF02	600 lf	$ 88,383	$ 135,756
Pipeline Improvement	Replace 4-inch pipelines constructed prior to 1960	FF03	13,567 lf	$ 1,998,484	$ 3,069,671
Pipeline Improvement	Replace 6-inch pipelines constructed prior to 1960	FF03	70,219 lf	$ 10,343,593	$ 15,887,759
Pump Station	Add two 1,750 Fire Flow Pumps to Zone 3 PS	FF04	2 ea	$ 500,000	$ 768,000
Peak Hour Improvement	Reconfigure Southwest Portion of Zone 1 Boundary	PH01	450 lf	$ 66,287	$ 101,817
Stanley Tank Maintenance	Prepare and recoat tank exterior	CIP01	1 ea	$ 195,313	$ 300,000
Elevated Tank Maintenance	Prepare and recoat top of tank exterior	CIP02	1 ea	$ 39,063	$ 60,000
Periodic Well Maintenance	Well Pump Removal and Rehabilitation	CIP03	1 L.S.	$ 50,000	$ 76,800
Water Storage Improvements	Install remote controlled/seismic shut-off valve at Reservoirs	CIP04	3 ea	$ 45,000	$ 69,120
		Total		**$ 13,373,260**	**$ 20,541,000**
		Construction Contingency (20%)		$ 2,674,652	
		Total Construction Cost		**$ 16,047,912**	
		Engineering (10%)		$ 1,604,791	
		Construction Management (10%)		$ 1,604,791	
		Program Implementation (8%)		$ 1,283,833	
		Total Existing System CIP Cost[(b)]		**$ 20,541,000**	

[(a)] Costs shown are based on January 2011 dollars and an ENR CCI of 8938 (20 Cities Average).
[(b)] Total cost rounded to nearest $1,000.

Table 9-4. Summary of Probable Construction Costs for Buildout System CIP[a]

Improvement Type	Improvement Description	CIP ID	Quantity	Estimated Construction Cost	CIP Cost (including contingency and cost allowances)
Buildout Capital Improvements					
Pipeline Improvement	Install 8-inch diameter pipeline in Dual Interest Area A	BDIA01	6,060 lf	$ 892,667	$ 1,371,136
Pipeline Improvement	Install 8-inch diameter pipeline in Dual Interest Area B	BDIA02	4,570 lf	$ 673,183	$ 1,034,009
Periodic Well Maintenance	Well Pump Removal and Rehabilitation	CIP05	3 L.S.	$ 150,000	$ 230,400
		Total		**$ 1,715,850**	**$ 2,636,000**
		Construction Contingency (20%)		$ 343,170	
		Total Construction Cost		**$ 2,059,020**	
		Engineering (10%)		$ 205,902	
		Construction Management (10%)		$ 205,902	
		Program Implementation (8%)		$ 164,722	
		Total Existing System CIP Cost[b]		**$ 2,636,000**	

[a] Costs shown are based on January 2011 dollars and an ENR CCI of 8938 (20 Cities Average).

[b] Total cost rounded to nearest $1,000.

9.4 RECOMMENDED CIP BY PROJECT TYPES

A summary of the costs of the recommended CIP by project type is provided in Table 9-5. As shown in Table 9-5, the total estimated recommended CIP cost for the City system is $23.18 million.

Table 9-5. Estimated Cost of Recommended CIP by Project Type

CIP Project Type	Existing System CIP Projects[a]	Buildout System CIP Projects
Pipelines	19.27	2.41
Storage Facility Maintenance	0.36	-
Water Storage Facility Improvements	0.07	
Pump Stations	0.77	-
Emergency Generators	-	-
Pressure Reducing Stations	-	-
Groundwater Well Maintenance	0.08	0.23
Total CIP Cost	$20.54 million	$2.64 million

9.5 CAPITAL IMPROVEMENT PROGRAM IMPLEMENTATION

As shown in Tables 9-3 and 9-5, several improvement projects are recommended for the existing system and the buildout system. The recommended improvements for the existing system should be completed within the next five years.

The construction of the improvements for the buildout system should be coordinated with the proposed schedules of future development to ensure that the required infrastructure will be in place to serve future customers.

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 9\FIG9-1_.mxd 8/25/2011

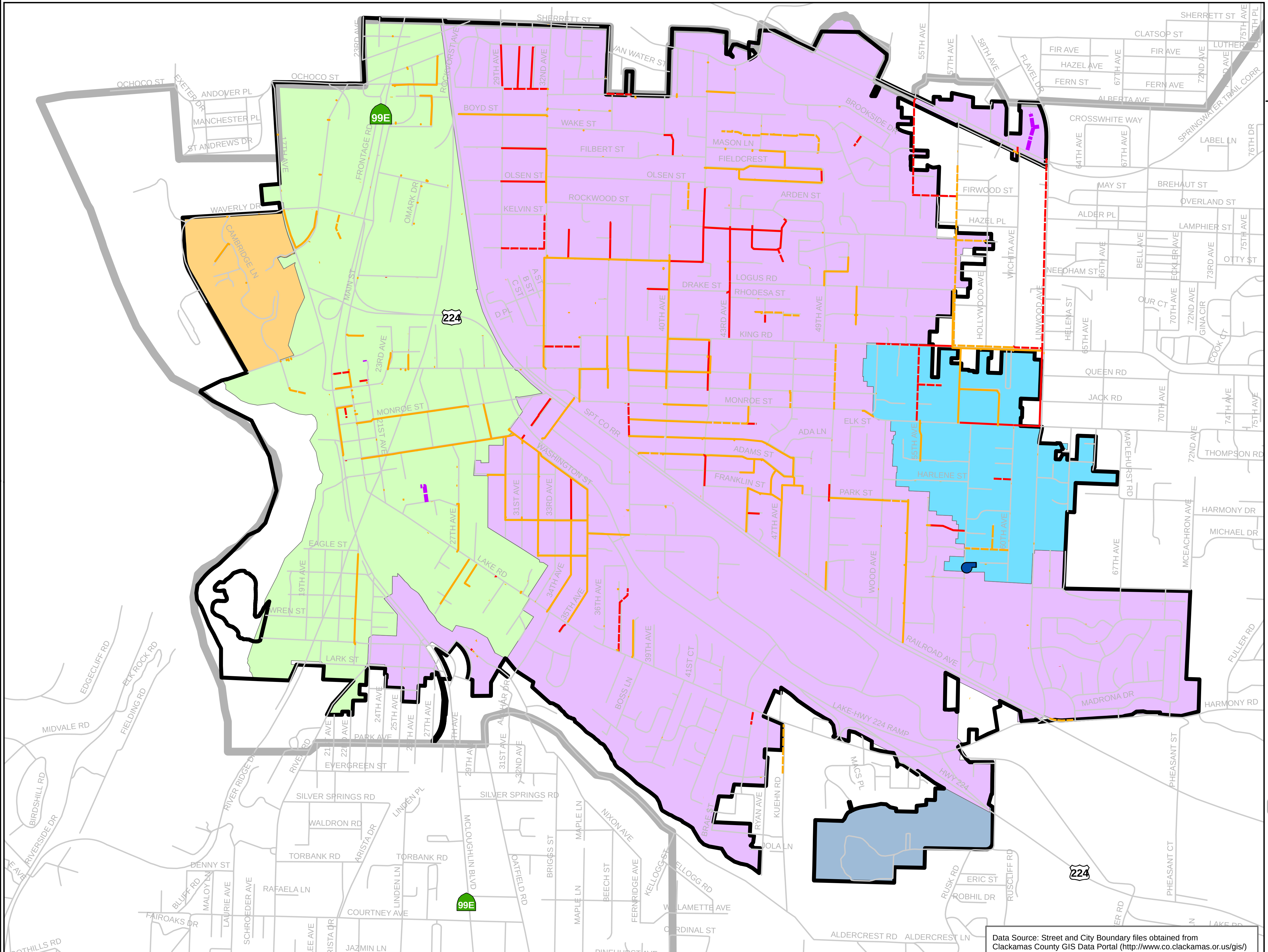

FIGURE 9-1

City of Milwaukie
2010 Water System Master Plan

EXISTING SYSTEM IMPROVEMENTS

N

0 750 1,500
Scale in Feet

LEGEND

- Public Area Fire Flow Improvements (FF01-FF02)

CIP Pipeline Improvements (FF03)

- 4-inch Diameter, Pre-1960
- 6-inch Diameter, Pre-1960
- 4-inch Diameter, Year Unknown
- 6-inch Diameter, Year Unknown
- Zone 3 Fire Flow Pumps

City's Pressure Zone Boundary

- Zone 1
- Zone 2
- Zone 3
- Zone 4
- Clackamas River Water
- City Boundary
- Urban Growth Boundary
- Street

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O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\New Chapter 9\FIG9-2_.mxd 8/25/2011

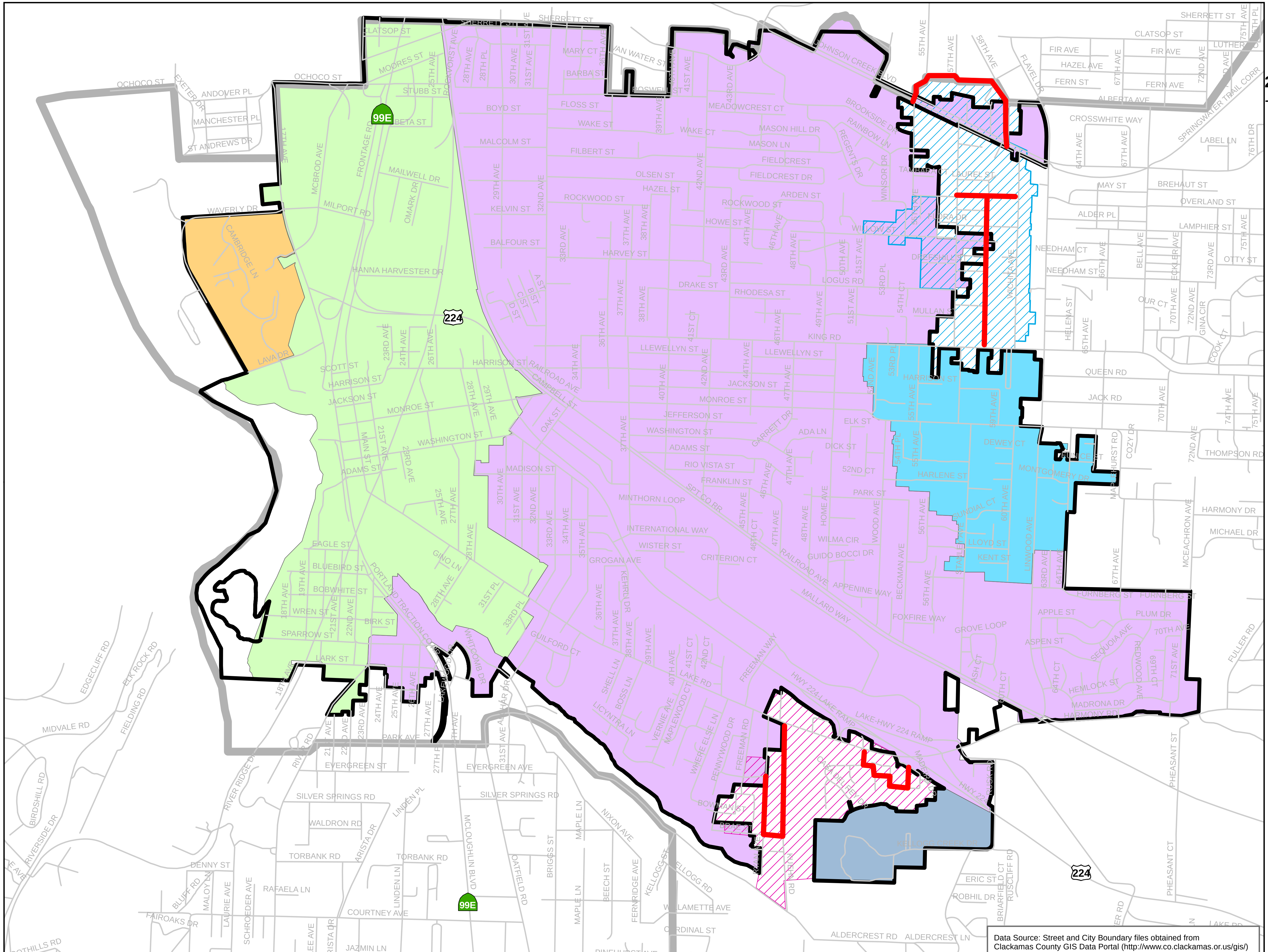

FIGURE 9-2

City of Milwaukie
2010 Water System Master Plan

BUILDOUT SYSTEM IMPROVEMENTS

N

0 750 1,500
Scale in Feet

LEGEND

City's Pressure Zone Boundary

- Zone 1
- Zone 2
- Zone 3
- Zone 4
- Clackamas River Water
- City Boundary
- Dual Interest Area A
- Dual Interest Area B
- Urban Growth Boundary
- Street
- 8-inch Diameter DI Pipeline

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APPENDIX A

Hydraulic Model Calibration – Hydrant Tests

HYDRANT (C-FACTOR) TESTS

Seventeen (17) tests were developed to confirm C-factors for the City of Milwaukie's water system, but only thirteen (13) tests were performed in the field on July 8 and 9, 2010. Four (4) hydrant tests were canceled due to constraint(s) identified by City staff. The selection of these hydrant tests was based on the location, size, material type, and age of the pipelines. These hydrant tests were used to evaluate pipeline friction factors (C-factors). As part of the model calibration process, these C-factors were adjusted, if necessary, to more closely represent actual observed field conditions.

Hydrant tests were simulated using West Yost's developed hydraulic model of the City's water system, and the preliminary C-factor values shown in Table 7-1. Results were then compared to actual field data to verify the preliminary C-factors and to determine the accuracy of the hydraulic model in replicating observed field pressures and flows. C-factors were then adjusted where necessary to minimize differences between static and residual hydrant pressures observed in the field to pressures simulated with the hydraulic model. The goal of the calibration effort was to achieve no greater than a 5 psi differential between the field hydrant test data and the model-simulated data.

Initial comparisons of model-simulated and field-observed data indicated possible issues associated with Hydrant Test No. 9. Additional discussion is presented later in this Appendix.

- Hydrant Test No. 9 – Measured and modeled pressures varied at all hydrants by up to 14 psi. Since the C-factor required for the model to simulate the ±5 psi pressure differential for Test No. 9 is unreasonable for this pipeline diameter, material and age, the results from the hydraulic model simulation indicate that for Test No. 9 there are either system configuration issues (*e.g.*, partially closed valve(s), inaccurate representation of pipeline connectivity or pipeline diameter) or there may have been an error with the residual pressure reading at the flowing hydrant. Additional field investigation to identify potential close valve is required. Two locations are identified based on the hydraulic results. Location 1 is along Montgomery Drive, east of Linwood. Location 2 is along Linwood, north of Furnberg Street.

The following sections describe each of the specific hydrant test locations and discuss a comparison between the model predicted pressures and the pressures observed in the field. A schematic describing the locations of the flowing and observed fire hydrants is also provided for each hydrant test location.

SUMMARY OF CALIBRATION RESULTS

Overall, the results of the hydrant tests generally validated the system pipeline configuration and confirmed preliminary C-factors. The average pressure differentials between those pressures observed in the field and those simulated by the model were within ±5 psi, except for 1 of the 13 hydrant tests that were performed. The results of the calibration runs from the hydrant tests performed indicated that the hydraulic model simulated the City's water system and was able to match field-observed pressures and flows.

The detailed results of individual calibration tests are provided in the following Tables A1 through A17.

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Hydrant Test No. 1

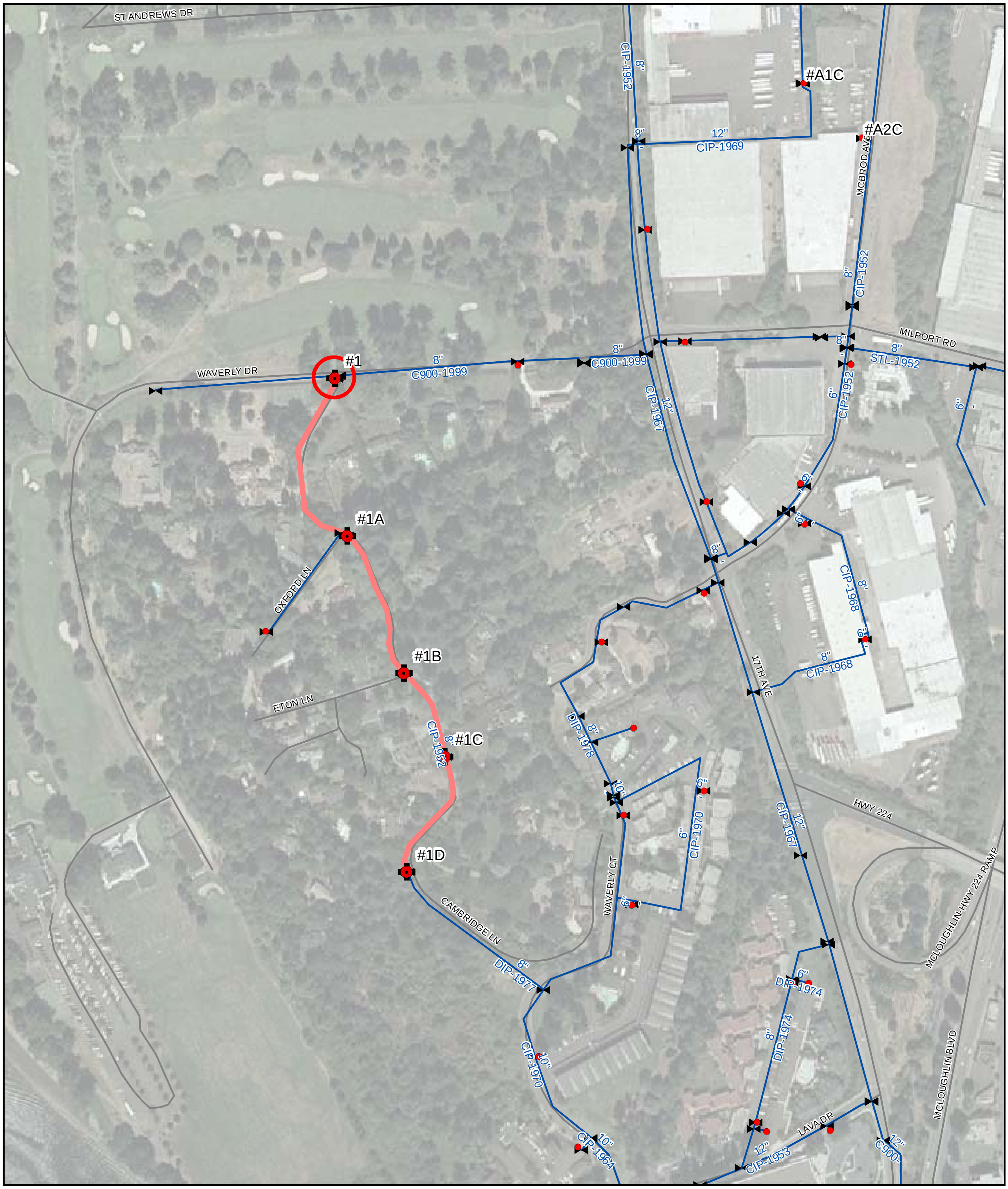
Hydrant Test No. 1 is located along Cambridge Lane, South of Wavery Drive. This test was intended to confirm the C-factor (initially assumed to equal 100) of an 8-inch diameter, CI pipeline constructed in 1952. This test was canceled due to constraint(s) identified by City staff.

Table A-1. Hydrant Test No. 1

Hydrant[1]	Field Data			Modeled Data			
	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	Comparison of Differential Pressures (psi) (g = c-f)
Flowing[2]							
1A[3]							
1B[4]							
1C[5]							
1D[6]							

CANCELED

(1) Location of fire hydrants can be found on Figure A-1.
(2) The "Flowing Hydrant" is located at Cambridge Lane, south of Waverly Drive.
(3) Hydrant 1A is located at Cambridge Lane, south of Oxford Lane.
(4) Hydrant 1B is located at Intersection of Cambridge Lane and Eton Lane.
(5) Hydrant 1C is located at Cambridge Lane, south of Test 1B.
(6) Hydrant 1D is located at Cambridge Lane, south of Test 1C.
NA = Not Applicable



LEGEND

- Hydrant
- Test Hydrant
- Valve
- Pipeline
- Test Pipeline

FIGURE A-1

City of Milwaukie
Water System Master Plan

TEST #1
(8" CIP 1952)

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Hydrant Test No. 2

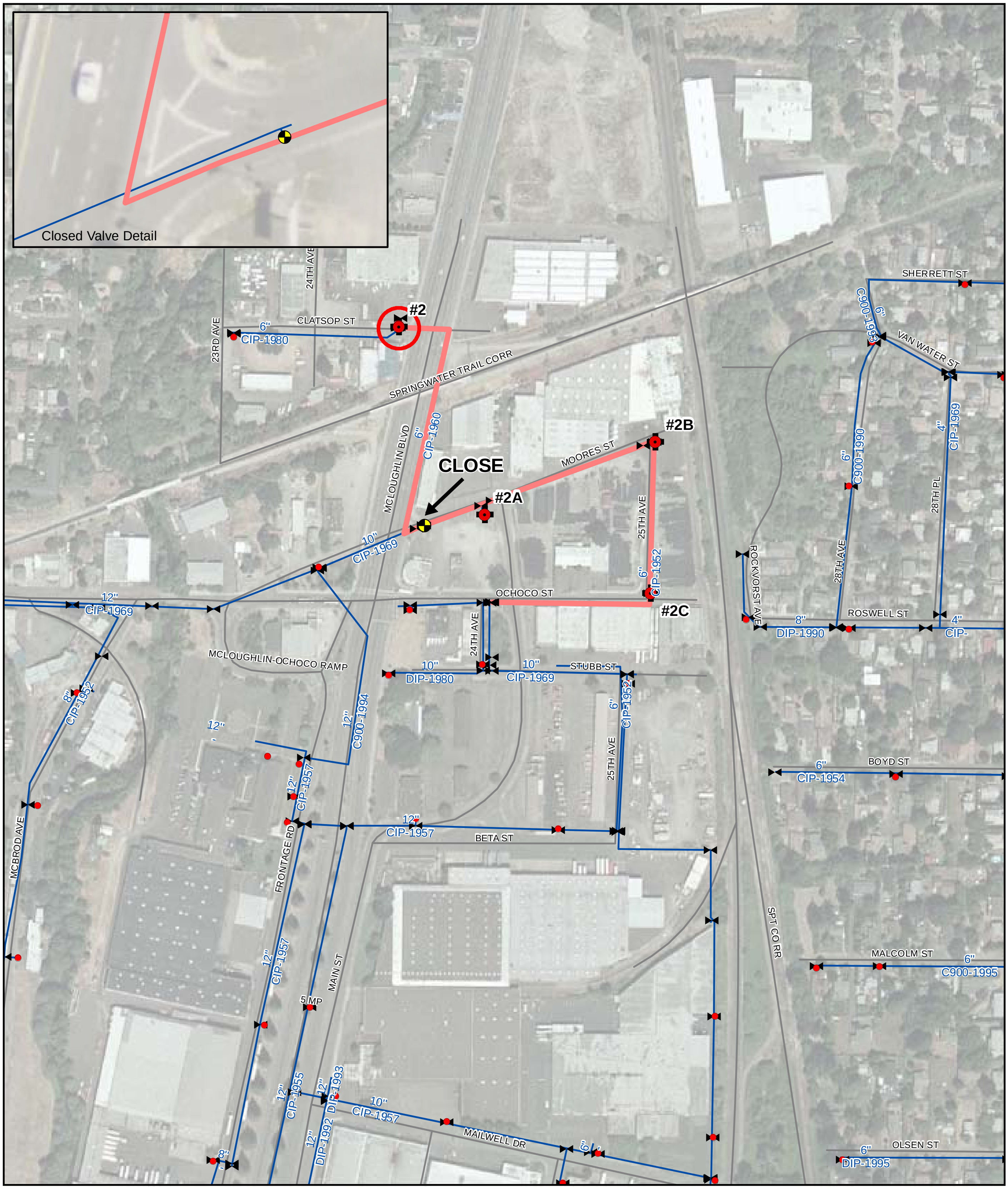
Hydrant Test No. 2 is located along Clatsop Street, west of McLoughlin Boulevard. This test was intended to confirm the C-factor (initially assumed to equal 100) of a 6-inch diameter, CI pipeline constructed in 1960. This test was canceled due to constraint(s) identified by City staff.

Table A-2. Hydrant Test No. 2

Hydrant[1]	Field Data			Modeled Data			
	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	Comparison of Differential Pressures (psi) (g = c-f)
Flowing[2]							
2A[3]							
2B[4]							
2C[5]							

CANCELED

(1) Location of fire hydrants can be found on Figure A-2.
(2) The “Flowing Hydrant” is located at Clatsop Street, west of McLoughlin Boulevard.
(3) Hydrant 2A is located at Moores Street, northeast of McLoughlin Boulevard.
(4) Hydrant 2B is located at Intersection of Moores Street and 25th Avenue.
(5) Hydrant 2C is located at 25th Avenue, north of Ochoco Street.
NA = Not Applicable



LEGEND

FIGURE A-2

City of Milwaukie
Water System Master Plan

TEST #2
(6" CIP 1980)

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O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\Hydrant Test\Fig_Test#2.mxd 6/14/2010

Hydrant Test No. 3

Hydrant Test No. 3 was performed along SE Mailwell Drive, east of SPT Corridor. This test was conducted to confirm the C-factor (initially assumed to be 130) of a 10-inch diameter, DI pipeline constructed in 1980. A comparison of the differential pressure readings predicted by the hydraulic model, compared to pressures actually measured in the field, demonstrates that the pressures predicted by the model are within ±5 psi of the measured field value. The calibrated model results and the field data are shown in Table A-3 and indicate that the use of a C-factor equal to 130 for this size and type of pipeline is valid.

Table A-3. Hydrant Test No. 3

Hydrant[1]	Field Data			Modeled Data			Comparison of Differential Pressures (psi) (g = c-f)
	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	
Flowing[2]	60	23		58	31		
3A[3]	60	36	24	58	38	20	4
3B[4]	63	42	21	58	42	16	5
3C[5]	59	44	15	58	45	12	3
3D[6]	59	50	9	58	50	8	1

(1) Location of fire hydrants can be found on Figure A-3.
(2) The "Flowing Hydrant" is located at SE Mailwell Drive, east of SPT Corridor.
(3) Hydrant 3A is located at Mailwell Drive, north of the southeast corner.
(4) Hydrant 3B is located at Mailwell Drive, north of Test 3A.
(5) Hydrant 3C is located at Mailwell Drive, north of Test 3B.
(6) Hydrant 3D is located at Mailwell Drive, north of Test 3C.
NA = Not Applicable

LEGEND

- Test Hydrant
- Hydrant
- Close Valve
- Valve
- Test Pipeline
- Pipeline

N

0 190 380

Scale in Feet

FIGURE A-3

City of Milwaukie
Water System Master Plan

TEST #3
(10" DIP 1980)

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Hydrant Test No. 4

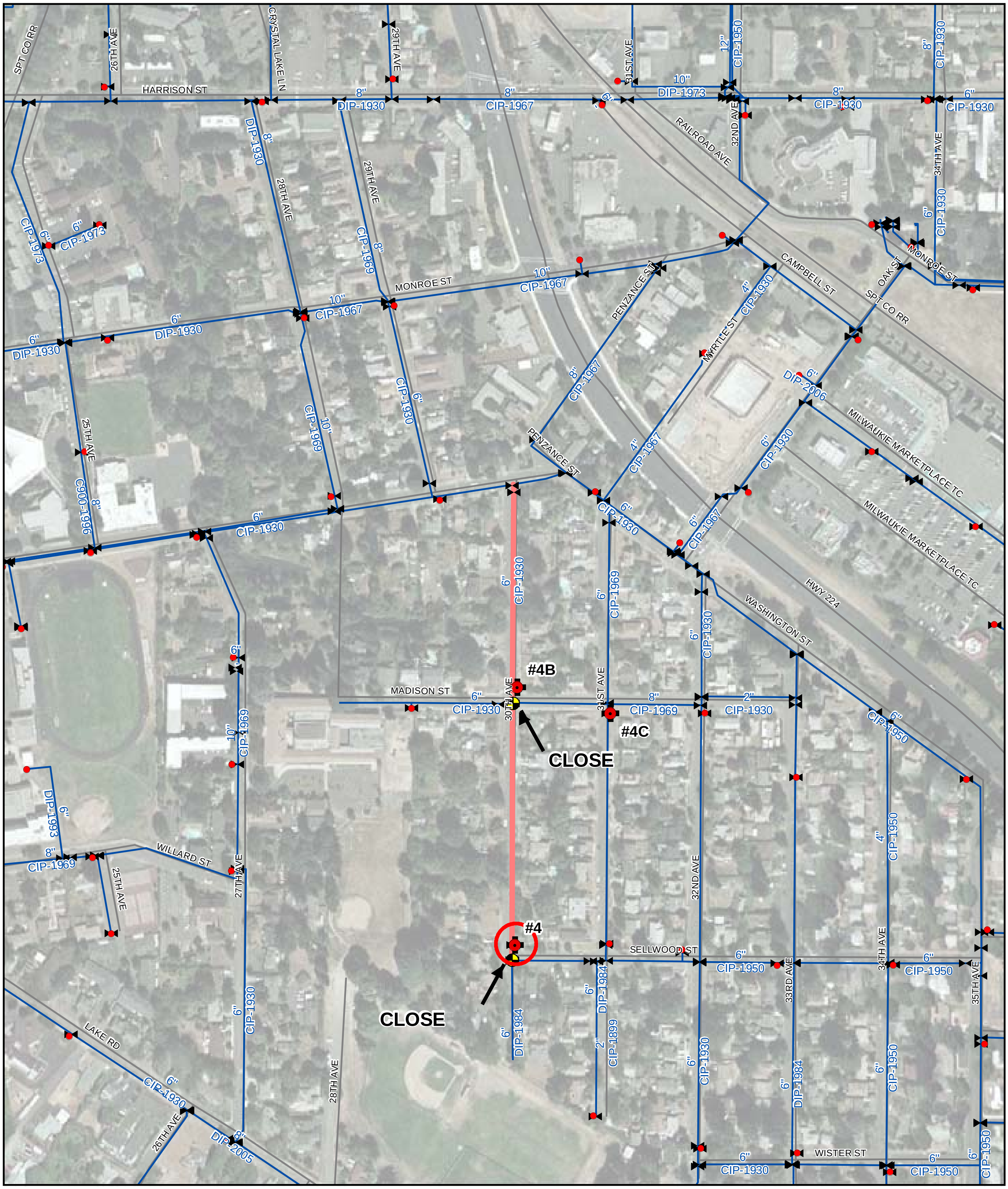
Hydrant Test No. 4 was performed at Madison Street, west of 30th Avenue. This test was conducted to confirm the C-factor (initially assumed to be 80) of a 6-inch diameter, CI pipeline constructed in 1930. During field testing, zone break was identified at north and south of hydrant test location. Consequently, the test location was rearranged, and observed hydrant 4A was eliminated.

A comparison of the differential pressure readings predicted by the hydraulic model, compared to pressures actually measured in the field, demonstrates that the pressures predicted by the model are within ±5 psi of the measured field value. The calibrated model results and the field data are shown in Table A-4A and indicate that the use of a C-factor equal to 80 for this size and type of pipeline is valid.

Table A-4A. Hydrant Test No. 4

	Field Data			Modeled Data			
Hydrant[1]	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	Comparison of Differential Pressures (psi) (g = c-f)
Flowing[2]	82	~0		66	6		
4A[3]	NA	NA	NA	NA	NA	NA	NA
4B[4]	78	43	35	81	47	34	1
4C[5]	85	80	5	81	76	5	0

(1) Location of fire hydrants can be found on Figure A-4.
(2) The "Flowing Hydrant" is located at Madison Street, west of 30th Avenue.
(3) Hydrant 4A was eliminated.
(4) Hydrant 4B is located at 30th Avenue, north of Madison Street.
(5) Hydrant 4C is located at SE Washington Street, east of 29th Avenue.
NA = Not Applicable



LEGEND

- Test Hydrant
- Hydrant
- Close Valve
- Valve
- Test Pipeline
- Pipeline

N

0 190 380

Scale in Feet

FIGURE A-4

City of Milwaukie
Water System Master Plan

TEST #4
(6" CIP 1930)

WEST YOST ASSOCIATES
Consulting Engineers

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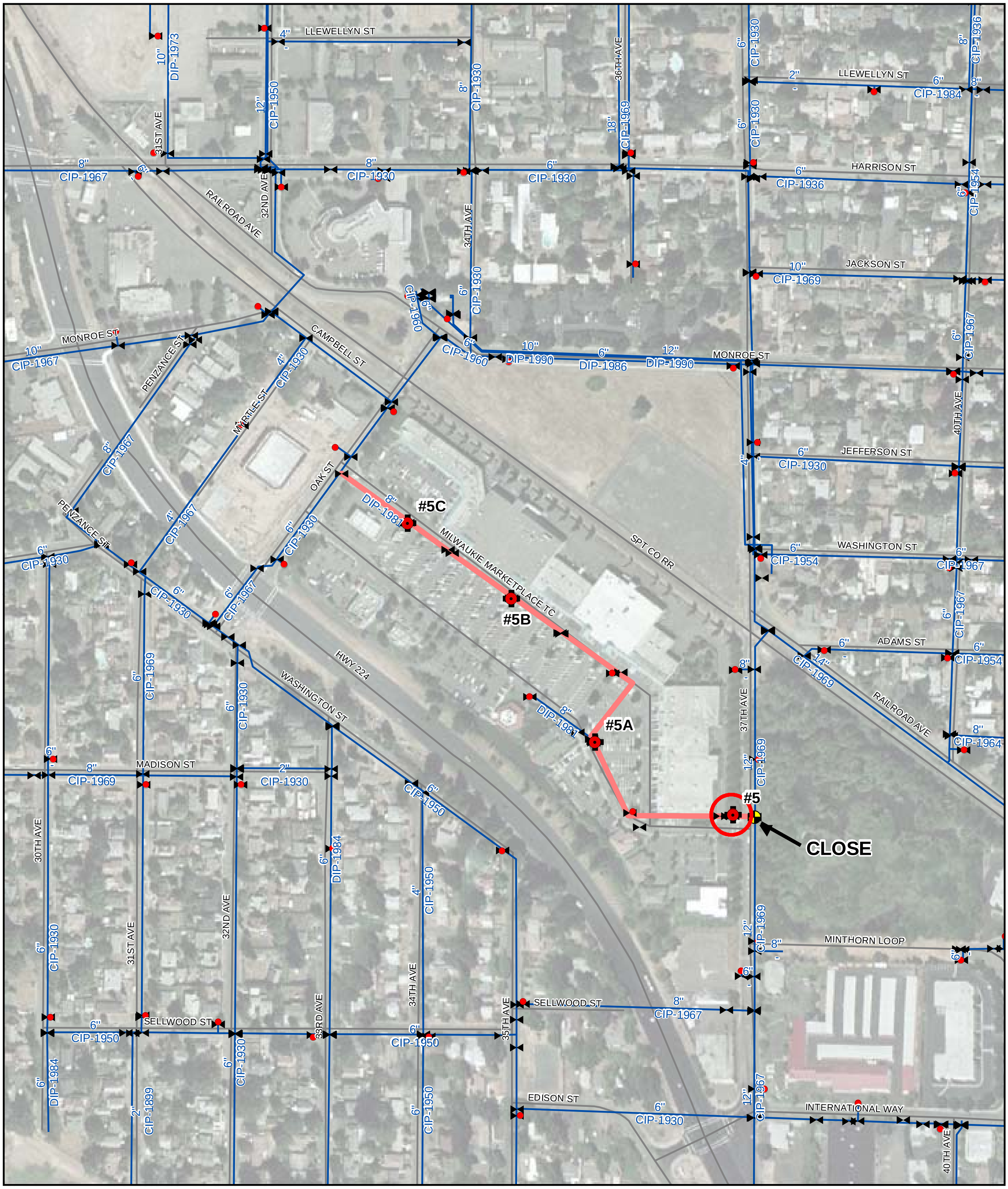
Hydrant Test No. 5

Hydrant Test No. 5 was performed along Milwaukie Marketplace. This test was conducted to confirm the C-factor (initially assumed to be 120) of an 8-inch diameter, DI pipeline constructed in 1981. A comparison of the differential pressure readings predicted by the hydraulic model, compared to pressures actually measured in the field, demonstrates that the pressures predicted by the model are within ±5 psi of the measured field value. This result indicates that the preliminary C-factor assigned to 8-inch DI pipelines was appropriate. The calibrated model results and the field data are shown in Table A-5A.

Table A-5A. Hydrant Test No. 5

	Field Data			Modeled Data			
Hydrant[1]	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	Comparison of Differential Pressures (psi) (g = c-f)
Flowing[2]	85	18		85	21		
5A[3]	85	28	57	84	29	56	1
5B[4]	80	37	43	84	39	45	-2
5C[5]	84	48	36	84	45	39	-3

(1) Location of fire hydrants can be found on Figure A-5.
(2) The “Flowing Hydrant” is located along Milwaukie Marketplace.
(3) Hydrant 5A is located at Milwaukie Marketplace, northwest of flowing hydrant.
(4) Hydrant 5B is located at Milwaukie Marketplace, northwest of Test 5A.
(5) Hydrant 5C is located at Milwaukie Marketplace, southeast of Oak Street.
NA = Not Applicable



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LEGEND

- Test Hydrant
- Hydrant
- Close Valve
- Valve
- Test Pipeline
- Pipeline

N

0 190 380

Scale in Feet

FIGURE A-5

City of Milwaukie
Water System Master Plan

TEST #5
(8" DIP 1981)

WEST YOST ASSOCIATES
Consulting Engineers

Hydrant Test No. 6

Hydrant Test No. 6 is located at along SE International Way. This test was intended to confirm the C-factor (initially assumed to equal 130) of a 12-inch diameter, DI pipeline constructed in 1979. A comparison of the differential pressure readings predicted by the hydraulic model, compared to pressures actually measured in the field, demonstrates that the pressures predicted by the model are within ±5 psi of the measured field value. The calibrated model results and the field data are shown in Table A-6 and indicate that the use of a C-factor equal to 130 for this size and type of pipeline is valid.

Table A-6. Hydrant Test No. 6

	Field Data			Modeled Data			
Hydrant[1]	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	Comparison of Differential Pressures (psi) (g = c-f)
Flowing[2]	88	50		86	61		
6A[3]	88	70	18	86	67	20	-2
6B[4]	80	67	13	83	67	17	-4
6C[5]	73	63	10	72	59	14	-4

(1) Location of fire hydrants can be found on Figure A-6.
(2) The “Flowing Hydrant” is located along SE International Way.
(3) Hydrant 6A is located at International Way, northwest of Freeman Way.
(4) Hydrant 6B is located at Freeman Way, southwest of International Way.
(5) Hydrant 6C is located at Freeman Way, north of Lake Road.
NA = Not Applicable

LEGEND

- Test Hydrant
- Hydrant
- Close Valve
- Valve
- Test Pipeline
- Pipeline

N

0 190 380

Scale in Feet

FIGURE A-6

City of Milwaukie
Water System Master Plan

TEST #6
(12" DIP 1979)

WEST YOST ASSOCIATES
Consulting Engineers

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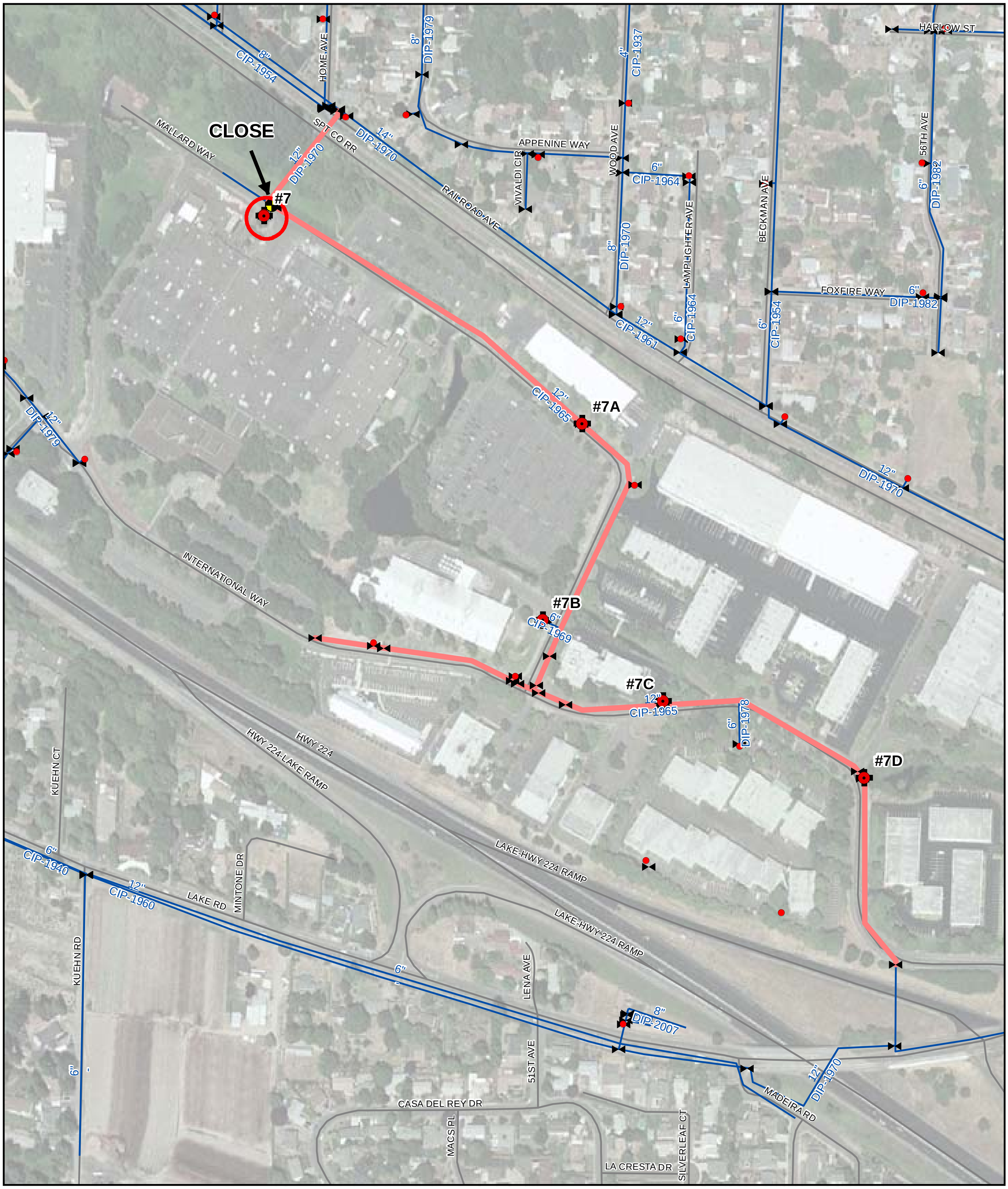
Hydrant Test No. 7

Hydrant Test No. 7 is located at Mallard Way, southeast of northwest end. This test was intended to confirm the C-factor (initially assumed to equal 120) of a 12-inch diameter, CI (assumed to be lined) pipeline constructed in 1965. A comparison of the differential pressure readings predicted by the hydraulic model, compared to pressures actually measured in the field, demonstrates that the pressures predicted by the model are within ±5 psi of the measured field value. The calibrated model results and the field data are shown in Table A-7 and indicate that the use of a C-factor equal to 120 for this size and type of pipeline is valid.

Table A-7. Hydrant Test No. 7

	Field Data			Modeled Data			
Hydrant[1]	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	Comparison of Differential Pressures (psi) (g = c-f)
Flowing[2]	87	48		86	59		
7A[3]	87	64	23	85	62	23	0
7B[4]	84	63	21	86	66	20	1
7C[5]	87	71	16	86	69	17	-1
7D[6]	89	74	15	87	73	14	1

(1) Location of fire hydrants can be found on Figure A-7.
(2) The "Flowing Hydrant" is located at Mallard Way, southeast of northwest end.
(3) Hydrant 7A is located at Mallard Drive, northwest of southeast corner.
(4) Hydrant 7B is located at Mallard Drive north of International Way.
(5) Hydrant 7C is located at International Way, east of Mallard Way.
(6) Hydrant 7C is located at International Way, southeast of Test 7C.
NA = Not Applicable



LEGEND

- Test Hydrant
- Hydrant
- Close Hydrant
- Valve
- Test Pipeline
- Pipeline

0 190 380

Scale in Feet

N

FIGURE A-7

City of Milwaukie
Water System Master Plan

TEST #7
(12" CIP 1965)

WEST YOST ASSOCIATES

Consulting Engineers

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Hydrant Test No. 8

Hydrant Test No. 8 was performed near Linwood Elementary School, north of Grove Loop. This test was conducted to confirm the C-factor (initially assumed to be 120) of a 10-inch diameter, CI (assumed to be lined) pipeline constructed in 1968. A comparison of the differential pressure readings predicted by the hydraulic model, compared to pressures actually measured in the field, demonstrates that the pressures predicted by the model are within ±5 psi of the measured field value. The calibrated model results and the field data are shown in Table A-8 and indicate that the use of a C-factor equal to 120 for this size and type of pipeline is valid.

Table A-8A. Hydrant Test No. 8

	Field Data			Modeled Data			
Hydrant[1]	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	Comparison of Differential Pressures (psi) (g = c-f)
Flowing[2]	56	19		55	15		
8A[3]	56	26	30	54	20	33	-3
8B[4]	54	25	29	54	27	27	2

(1) Location of fire hydrants can be found on Figure A-8.
(2) The “Flowing Hydrant” is located near Linwood Elementary School, north of Grove Loop.
(3) Hydrant 8A is located near Linwood Elementary, southeast of flowing hydrant.
(4) Hydrant 8B is located at near Linwood Elementary, west of Linwood Avenue.
NA = Not Applicable

LEGEND

- Test Hydrant
- Hydrant
- Close Valve
- Valve
- Test Pipeline
- Pipeline

0 190 380
Scale in Feet

FIGURE A-8

City of Milwaukie
Water System Master Plan

TEST #8
(10" CIP 1968)

WEST YOST ASSOCIATES
Consulting Engineers

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Hydrant Test No. 9

Hydrant Test No. 9 was performed on 66th Avenue, north of Eunice Street. This test was conducted to confirm the C-factor (initially assumed to equal 120) of a 6-inch diameter, DI pipeline constructed in 1985. A comparison of the differential pressure readings predicted by the hydraulic model, compared to pressures actually measured in the field, demonstrates that the pressures predicted by the model are not within ±5 psi of the measured field value. The calibrated model results and the field data are shown in Table A-9.

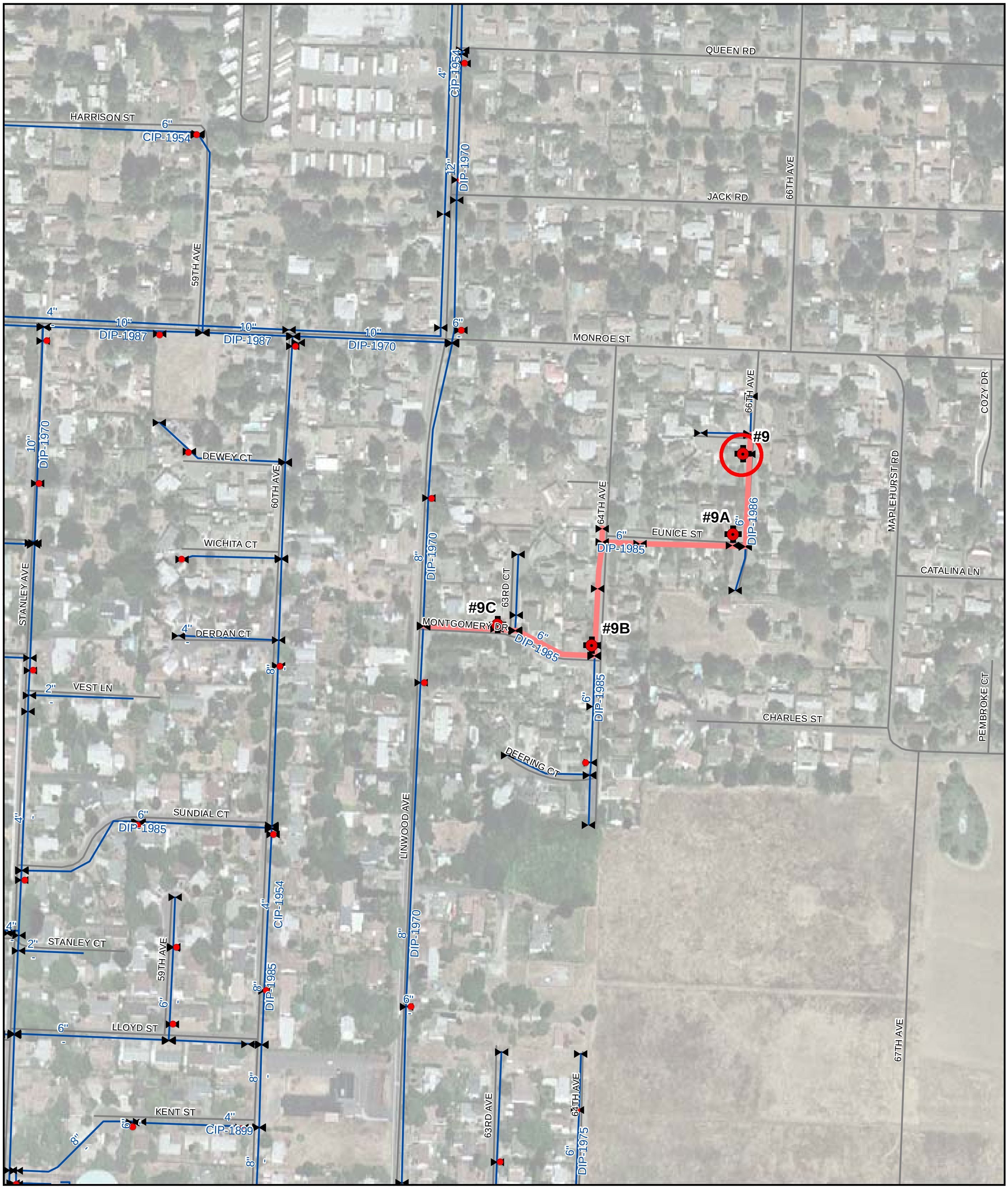
Since the C-factor required for the model to simulate the ±5 psi pressure differential for Test No. 9 is unreasonable for this pipeline diameter, material and age, the results from the hydraulic model simulation indicate that for Test No. 9 there are either system configuration issues (*e.g.*, partially closed valve(s), inaccurate representation of pipeline connectivity or pipeline diameter). West Yost Associates has identified potential partially closed valve in the vicinity of Test No. 9. There are two locations that require additional field investigation for potential close valve. Location 1 is along Montgomery Drive, east of Linwood. Location 2 is along Linwood, north of Furnberg Street.

For Test 9, it is recommended that City staff first confirm the valve status on the two location identified by West Yost. When the partial closed valve on these two locations were assumed in the hydraulic model, Test 9 simulates within a 5 psi differential from the field hydrant test data. This result indicates that the preliminary C-factor of 120 that was assigned to 6-inch DI pipelines was appropriate.

Table A-9. Hydrant Test No. 9

Hydrant(1)	Field Data			Modeled Data			Comparison of Differential Pressures (psi) (g = c-f)
	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	
Flowing(2)	65	7		71	0		
9A(3)	65	12	53	68	1	67	-14
9B(4)	69	34	35	75	40	35	0
9C(5)	67	43	24	71	51	20	4

(1) Location of fire hydrants can be found on Figure A-9.
(2) The "Flowing Hydrant" is located at 66th Avenue, north of Eunice Street.
(3) Hydrant 9A is located at Eunice Street, west of 66th Avenue.
(4) Hydrant 9B is located at 64th Avenue, north of Montgomery Drive.
(5) Hydrant 9C is located at Montgomery Drive, west of 63rd Court.
NA = Not Applicable



LEGEND

- Test Hydrant
- Hydrant
- Valve
- Test Pipeline
- Pipeline

N

0 190 380

Scale in Feet

FIGURE A-9

City of Milwaukie
Water System Master Plan

TEST #9
(6" DIP 1985)

WEST YOST ASSOCIATES
Consulting Engineers

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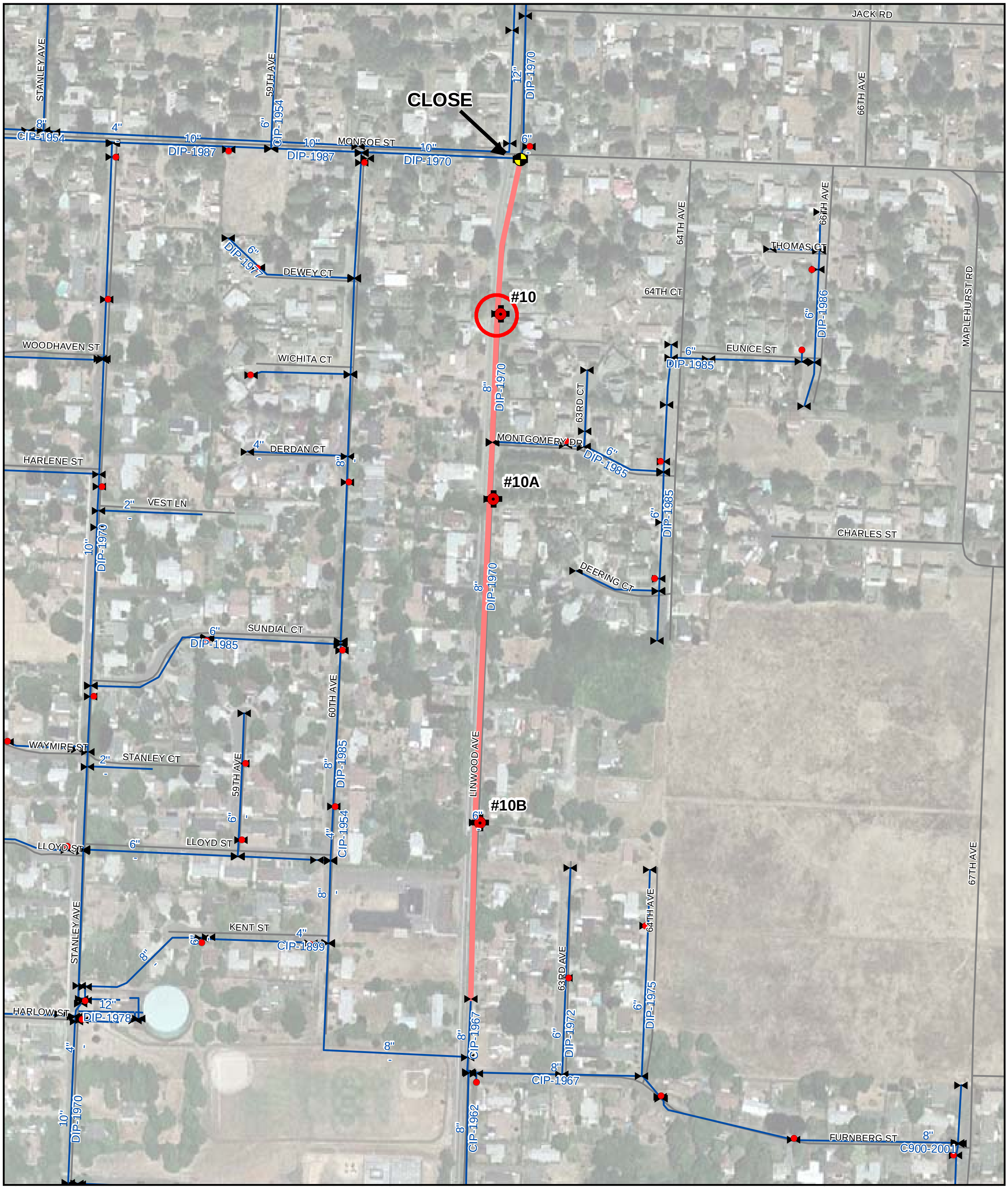
Hydrant Test No. 10

Hydrant Test No. 10 was performed on Linwood Avenue, north of Montgomery drive. This test was conducted to confirm the C-factor (initially assumed to equal 110) of an 8-inch diameter, DI pipeline constructed in 1970. A comparison of the differential pressure readings predicted by the hydraulic model, compared to pressures actually measured in the field, demonstrates that the pressures predicted by the model are within ±5 psi of the measured field value. The calibrated model results and the field data are shown in Table A-10 and indicate that the use of a C-factor equal to 110 for this size and type of pipeline is valid.

Table A-10. Hydrant Test No. 10

Hydrant[1]	Field Data			Modeled Data			Comparison of Differential Pressures (psi) (g = c-f)
	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	
Flowing[2]	65	10		71	12		
10A[3]	70	23	47	70	21	48	-1
10B[4]	71	36	35	75	44	31	4

(1) Location of fire hydrants can be found on Figure A-10.
(2) The “Flowing Hydrant” is located at Linwood Avenue, north of Montgomery drive.
(3) Hydrant 10A is located at Linwood Avenue, south of Montgomery Drive.
(4) Hydrant 10B is located at Linwood Avenue, north of Furnberg Street.
NA = Not Applicable



LEGEND

- Test Hydrant
- Hydrant
- Close Valve
- Valve
- Test Pipeline
- Pipeline

0 190 380
Scale in Feet

N

FIGURE A-10

City of Milwaukie
Water System Master Plan

TEST #10
(8" DIP 1970)

WEST YOST ASSOCIATES
Consulting Engineers

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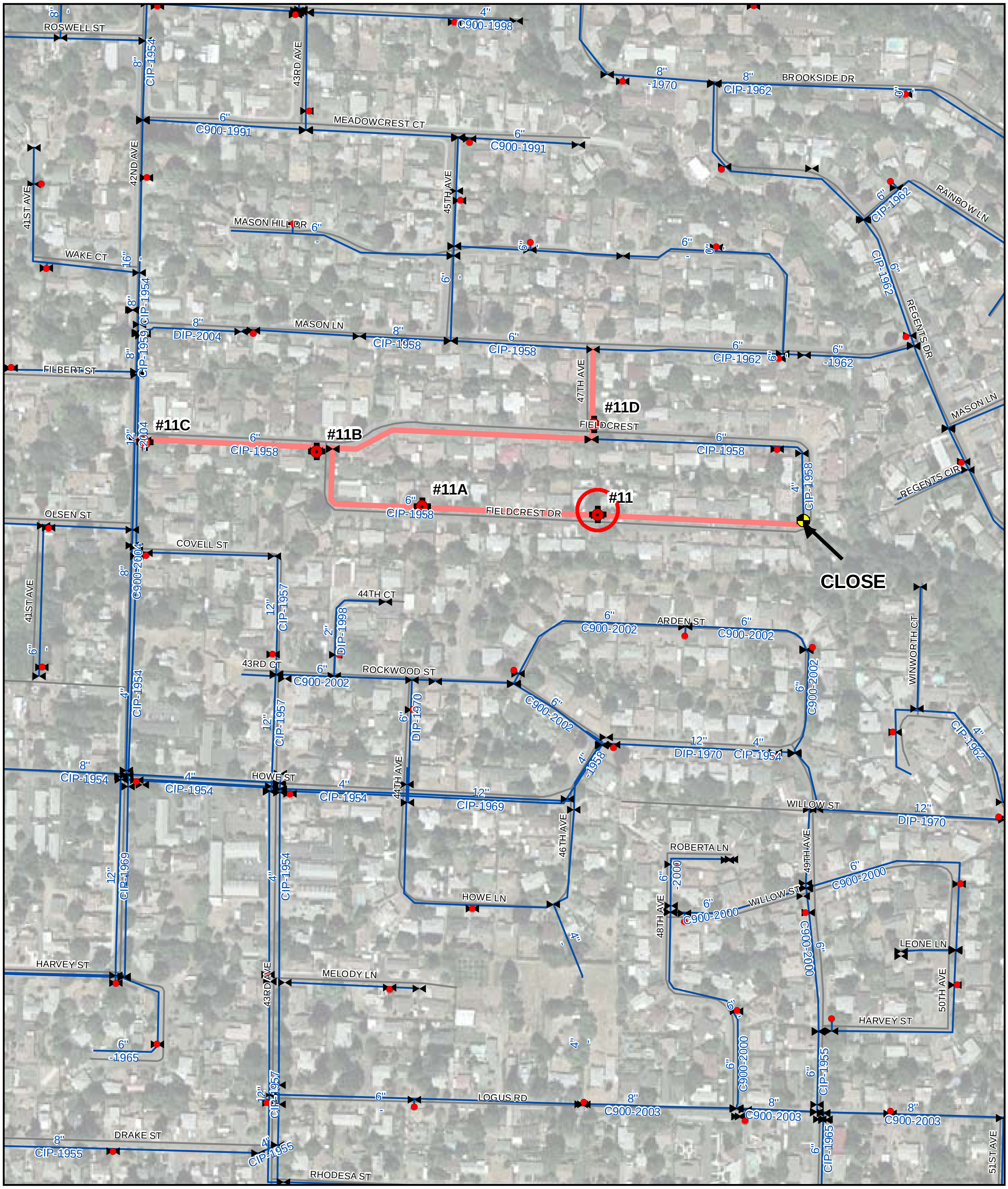
Hydrant Test No. 11

Hydrant Test No. 11 was performed on Middle of Fieldcrest Drive. This test was conducted to confirm the C-factor (initially assumed to equal 100) of a 6-inch diameter, CI pipeline constructed in 1958. A comparison of the differential pressure readings predicted by the hydraulic model, compared to pressures actually measured in the field, demonstrates that the pressures predicted by the model are within ±5 psi of the measured field value. The calibrated model results and the field data are shown in Table A-11 and indicate that the use of a C-factor equal to 100 for this size and type of pipeline is valid.

Table A-11. Hydrant Test No. 11

Hydrant[1]	Field Data			Modeled Data			Comparison of Differential Pressures (psi) (g = c-f)
	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	
Flowing[2]	56	8		55	2		
11A[3]	56	27	29	54	22	32	-3
11B[4]	55	38	17	54	40	14	3
11C[5]	56	50	6	55	50	5	1
11D[6]	54	48	6	55	47	8	-2

(1) Location of fire hydrants can be found on Figure A-11.
(2) The "Flowing Hydrant" is located at Middle of Fieldcrest Drive.
(3) Hydrant 11A is located at Fieldcrest Drive, west of flowing hydrant.
(4) Hydrant 11B is located at Fieldcrest Street, west of Fieldcrest Drive.
(5) Hydrant 11C is located at Fieldcrest Street, east of 42nd Avenue.
(6) Hydrant 11D is located at 47th Avenue, north of Fieldcrest Street.
NA = Not Applicable



LEGEND

- Test Hydrant
- Hydrant
- Close Valve
- Valve
- Test Pipeline
- Pipeline

0 190 380
Scale in Feet

N

FIGURE A-11

City of Milwaukie
Water System Master Plan

TEST #11
(6" CIP 1958)

WEST YOST ASSOCIATES
Consulting Engineers

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Hydrant Test No. 12

Hydrant Test No. 12 was performed on Filbert Street, east of 32nd Avenue. This test was conducted to confirm the C-factor (initially assumed to equal 110) of an 8-inch diameter, CI pipeline constructed in 1969. A comparison of the differential pressure readings predicted by the hydraulic model, compared to pressures actually measured in the field, demonstrates that the pressures predicted by the model are within ±5 psi of the measured field value. The calibrated model results and the field data are shown in Table A-12 and indicate that the use of a C-factor equal to 110 for this size and type of pipeline is valid.

Table A-12. Hydrant Test No. 12

	Field Data			Modeled Data			
Hydrant(1)	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	Comparison of Differential Pressures (psi) (g = c-f)
Flowing(2)	58			58	38		
12A(3)	58	45	13	55	40	14	-1
12B(4)	50	43	7	55	46	9	-2
12C(5)	56	48	8	55	47	8	0
12D(6)	52	45	7	51	44	8	-1

(1) Location of fire hydrants can be found on Figure A-12.
(2) The "Flowing Hydrant" is located at Filbert Street, east of 32nd Avenue.
(3) Hydrant 12A is located at Filbert Street, east of flowing hydrant.
(4) Hydrant 12B is located at Filbert Street, east of Test 12A.
(5) Hydrant 12C is located at Filbert Street, east of Test 12B.
(6) Hydrant 12D is located at Olsen Street.
NA = Not Applicable

LEGEND

- Test Hydrant
- Hydrant
- Close Valve
- Valve
- Test Pipeline
- Pipeline

N

0 190 380

Scale in Feet

FIGURE A-12

City of Milwaukie Water System Master Plan

TEST #12 (8" CIP 1969)

WEST YOST ASSOCIATES Consulting Engineers

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Hydrant Test No. 13

Hydrant Test No. 13 is located at Sherrett Street, west of 29th Avenue. This test was intended to confirm the C-factor (initially assumed to equal 140) of a 6-inch diameter, PVC pipeline constructed in 1993. A comparison of the differential pressure readings predicted by the hydraulic model, compared to pressures actually measured in the field, demonstrates that the pressures predicted by the model are within ±5 psi of the measured field value. The calibrated model results and the field data are shown in Table A-13 and indicate that the use of a C-factor equal to 140 for this size and type of pipeline is valid.

Table A-13. Hydrant Test No. 13

	Field Data			Modeled Data			
Hydrant(1)	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	Comparison of Differential Pressures (psi) (g = c-f)
Flowing(2)	62	10		59	11		
13A(3)	62	23	39	60	24	36	3
13B(4)	58	29	29	61	33	28	1

(1) Location of fire hydrants can be found on Figure A-13.
(2) The “Flowing Hydrant” is located at Sherrett Street, west of 29th Avenue.
(3) Hydrant 13A is located at 28th Avenue, south of Van Water Street.
(4) Hydrant 13B is located at 28th Avenue, north of Rosewell Street.
NA = Not Applicable

LEGEND

- Test Hydrant
- Hydrant
- Close Valve
- Valve
- Test Pipeline
- Pipeline

0 190 380

Scale in Feet

FIGURE A-13

City of Milwaukie
Water System Master Plan

TEST #13
(6" PVC 1993)

WEST YOST ASSOCIATES
Consulting Engineers

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Hydrant Test No. A1

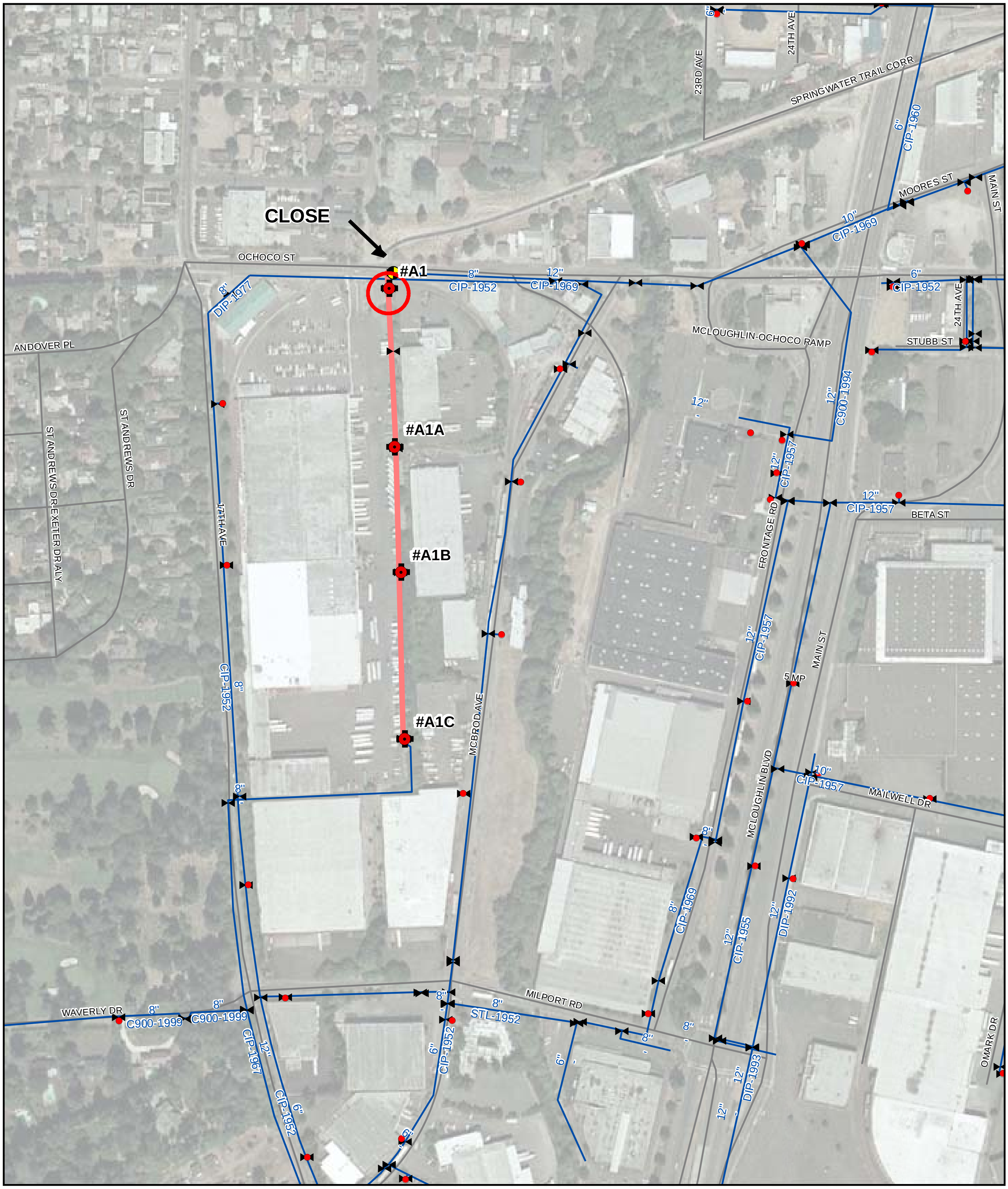
Hydrant Test No. A1 is located at between 17th Avenue and McBrod Avenue, south of Ochoco Street. This test was intended to confirm the C-factor (initially assumed to equal 120) of a 12-inch diameter, CI pipeline constructed in 1969. This alternate hydrant test was not performed due to constraint(s) identified by City staff.

Table A-14. Hydrant Test No. A1

CANCELED

Hydrant	Field Data			Modeled Data			Comparison of Differential Pressures (psi) (g = c-f)
	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	
Flowing[2]							
A1A[3]							
A1B[4]							
A1C[5]							

[1] Location of fire hydrants can be found on Figure A-14.
[2] The "Flowing Hydrant" is located at between 17th Avenue and McBrod Avenue, south of Ochoco Street.
[3] Hydrant A1-A is located at between 17th Avenue and McBrod Avenue, south of flowing hydrant.
[4] Hydrant A1-B is located at between 17th Avenue and McBrod Avenue, south of Test A1A.
[5] Hydrant A1-C is located at between 17th Avenue and McBrod Avenue, south of Test A1B.
NA = Not Applicable



LEGEND

- Test Hydrant
- Hydrant
- Close Valve
- Valve
- Test Pipeline
- Pipeline

0 190 380

Scale in Feet

FIGURE A-A1

City of Milwaukie Water System Master Plan

TEST #A1 (12" CIP 1969)

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Hydrant Test No. A2

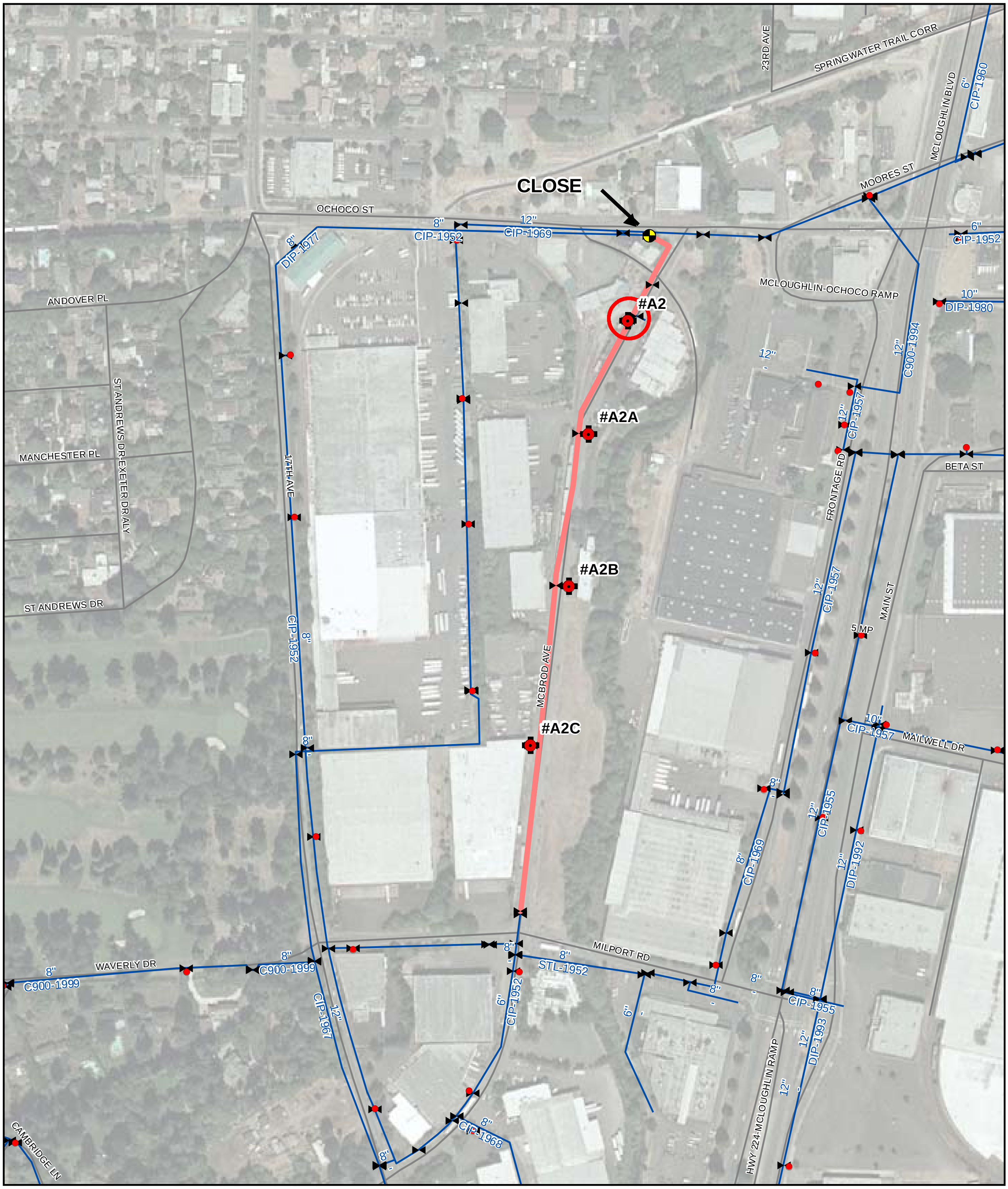
Hydrant Test No. A2 is located at McBrod Avenue, south of Ochoco Street. This test was intended to confirm the C-factor (initially assumed to equal 100) of an 8-inch diameter, CI pipeline constructed in 1952. This alternate hydrant test was not performed due to constraint(s) identified by City staff.

Table A-15. Hydrant Test No. A2

CANCELED

Hydrant[1]	Field Data			Modeled Data			Comparison of Differential Pressures (psi) (g = c-f)
	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	
Flowing[2]							
A2A[3]							
A2B[4]							
A2C[5]							

(1) Location of fire hydrants can be found on Figure A-15.
(2) The "Flowing Hydrant" is located at McBrod Avenue, south of Ochoco Street.
(3) Hydrant A-2A is located at McBrod Avenue, south of flowing hydrant.
(4) Hydrant A-2B is located at McBrod Avenue, south of Test A2A.
(5) Hydrant A-2C is located at McBrod Avenue, south of Test A2B.
NA = Not Applicable



LEGEND

- Test Hydrant
- Hydrant
- Close Valve
- Valve
- Test Pipeline
- Pipeline

0 190 380

Scale in Feet

FIGURE A-A2

City of Milwaukie
Water System Master Plan

TEST #A2
(8" CIP 1952)

WEST YOST ASSOCIATES
Consulting Engineers

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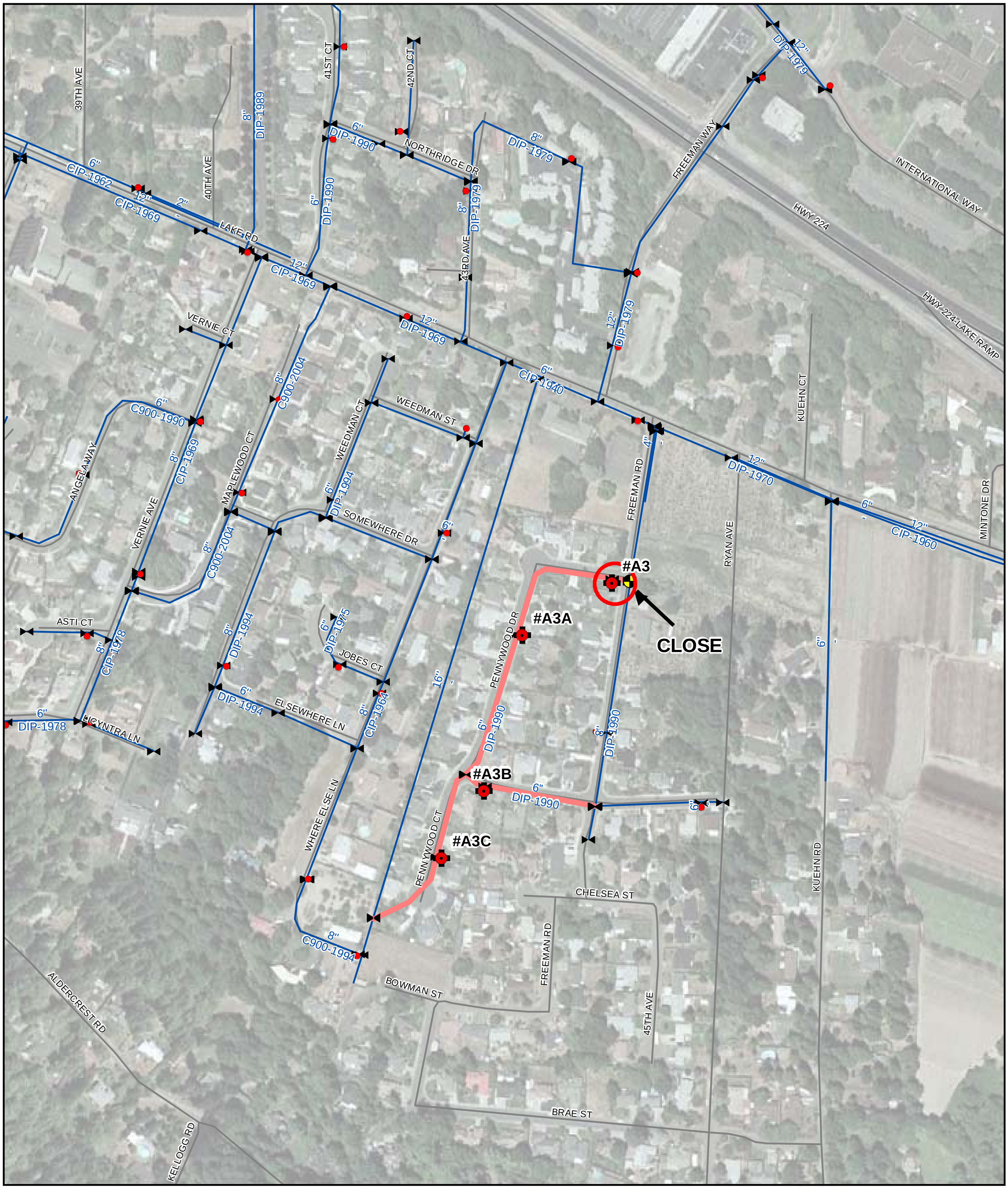
Hydrant Test No. A3

Hydrant Test No. A3 was performed along Pennywood Drive, west of Freeman Road. This test was conducted to confirm the C-factor (initially assumed to equal 120) of a 6-inch diameter, DI pipeline constructed in 1990. A comparison of the differential pressure readings predicted by the hydraulic model, compared to pressures actually measured in the field, demonstrates that the pressures predicted by the model are within ±5 psi of the measured field value. The calibrated model results and the field data are shown in Table A-16 and indicate that the use of a C-factor equal to 120 for this size and type of pipeline is valid.

Table A-16. Hydrant Test No. A3

Hydrant(1)	Field Data			Modeled Data			Comparison of Differential Pressures (psi) (g = c-f)
	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	
Flowing(2)	84	15		78	21		
A-3A(3)	84	42	42	81	42	39	3
A-3B(4)	79	60	19	82	64	18	1
A-3C(5)	85	69	16	83	66	17	-1

(1) Location of fire hydrants can be found on Figure A-16.
(2) The "Flowing Hydrant" is located at Pennywood Drive, west of Freeman Road.
(3) Hydrant A-3A is located at Pennywood Drive, north of Pennywood Court.
(4) Hydrant A-3B is located at Pennywood Drive, east of Pennywood Court.
(5) Hydrant A-3C is located at Pennywood Court, south of Pennywood Drive.
NA = Not Applicable



LEGEND

- Test Hydrant
- Hydrant
- Close Valve
- Valve
- Test Pipeline
- Pipeline

0 190 380
Scale in Feet

FIGURE A-A3

City of Milwaukie
Water System Master Plan

TEST #A3
(6" DIP 1990)

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O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\Hydrant Test\Fig_Test#A3.mxd 6/14/2010

Hydrant Test No. A4

Hydrant Test No. A4 is located along Johnson Creek Boulevard, southeast of 45th Place. This test was intended to confirm the C-factor (initially assumed to equal 110) of an 8-inch diameter, DI pipeline constructed in 1970. A comparison of the differential pressure readings predicted by the hydraulic model, compared to pressures actually measured in the field, demonstrates that the pressures predicted by the model are within ±5 psi of the measured field value, except for Hydrant A-4A. The calibrated model results and the field data are shown in Table A-17.

The C-factor for this type of pipeline was adjusted to 100. Results indicate that the pressures predicted by the model are within ±5 psi of the measured field value.

Table A-17. Hydrant Test No. A4

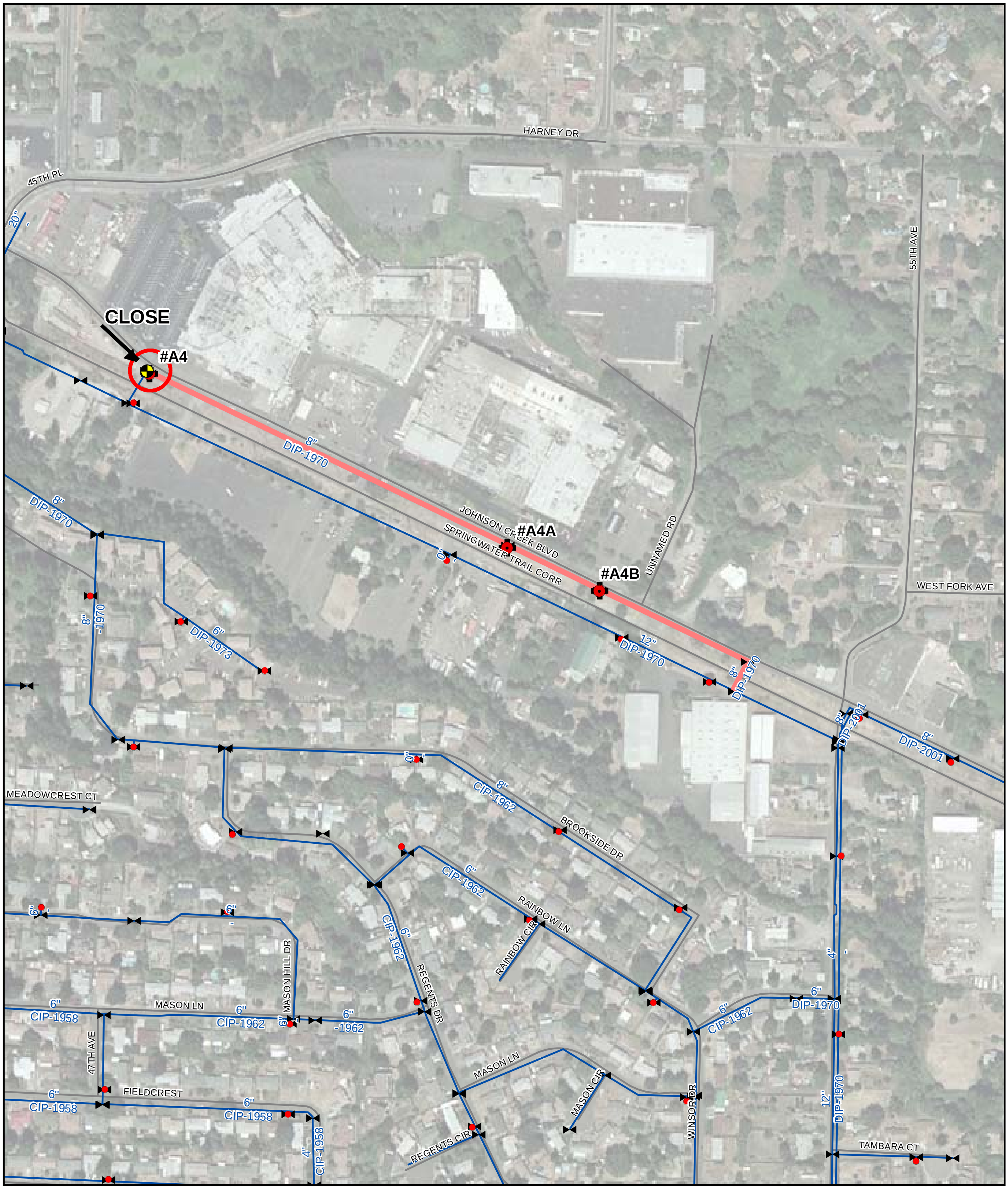
	Field Data			Modeled Data			
Hydrant[1]	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	Comparison of Differential Pressures (psi) (g = c-f)
Flowing[2]	77	40		77	47		
A-4A[3]	77	47	30	72	56	16	14
A-4B[4]	67	49	18	70	59	12	-2

(1) Location of fire hydrants can be found on Figure A-17.
(2) The "Flowing Hydrant" is located at Johnson Creek Boulevard, southeast of 45th Place.
(3) Hydrant A-4A is located at Johnson Creek Boulevard, southeast of flowing hydrant.
(4) Hydrant A-4B is located at Johnson Creek Boulevard, southeast of Test A4A.
NA = Not Applicable

Table A-17B. Hydrant Test No. A4 (Comparison for C-factor = 100)

	Field Data			Modeled Data			
Hydrant[1]	Static Pressure (psi) (a)	Residual Pressure (psi) (b)	Differential Pressure (c = a-b)	Static Pressure (psi) (d)	Residual Pressure (psi) (e)	Differential Pressure (psi) (f = d-e)	Comparison of Differential Pressures (psi) (g = c-f)
Flowing[2]	77	40		77	36		
A-4A[3]	77	47	30	72	48	25	5
A-4B[4]	67	49	18	70	51	20	-2

(1) Location of fire hydrants can be found on Figure A-17.
(2) The "Flowing Hydrant" is located at Johnson Creek Boulevard, southeast of 45th Place.
(3) Hydrant A-4A is located at Johnson Creek Boulevard, southeast of flowing hydrant.
(4) Hydrant A-4B is located at Johnson Creek Boulevard, southeast of Test A4A.
NA = Not Applicable



O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\Hydrant Test\Fig_Test#A4.mxd 6/14/2010

LEGEND

- Test Hydrant
- Hydrant
- Close Valve
- Valve
- Test Pipeline
- Pipeline

N

0 190 380

Scale in Feet

FIGURE A-A4

City of Milwaukie Water System Master Plan

TEST #A4 (8" DIP 1970)

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APPENDIX B

HPR Locations and Verification Results

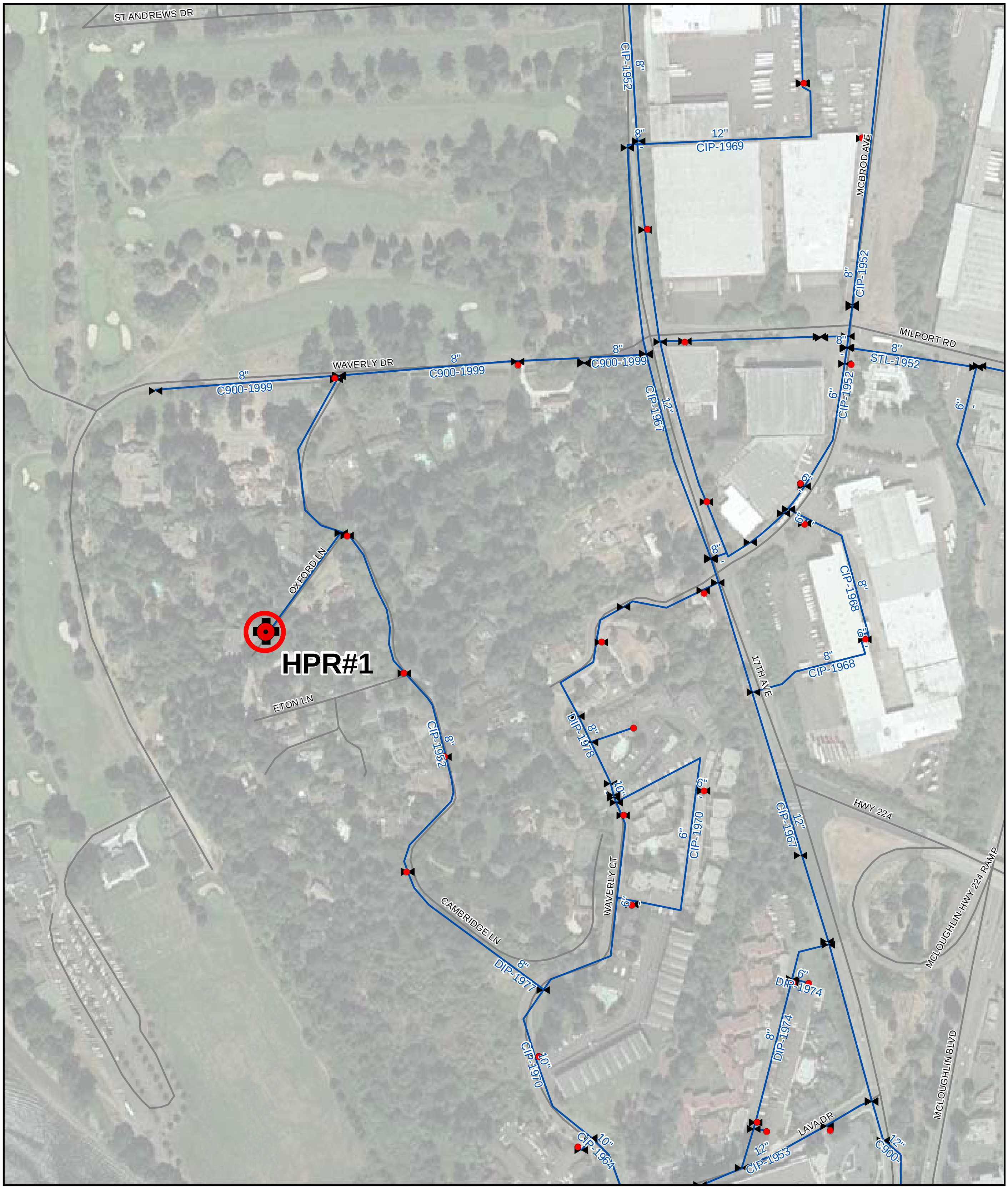

FIGURE B-1

LEGEND

- Hydrant
- HPR Location
- Valve
- Pipeline

N

0 190 380

Scale in Feet

City of Milwaukie
Water System Master Plan

HPR #1

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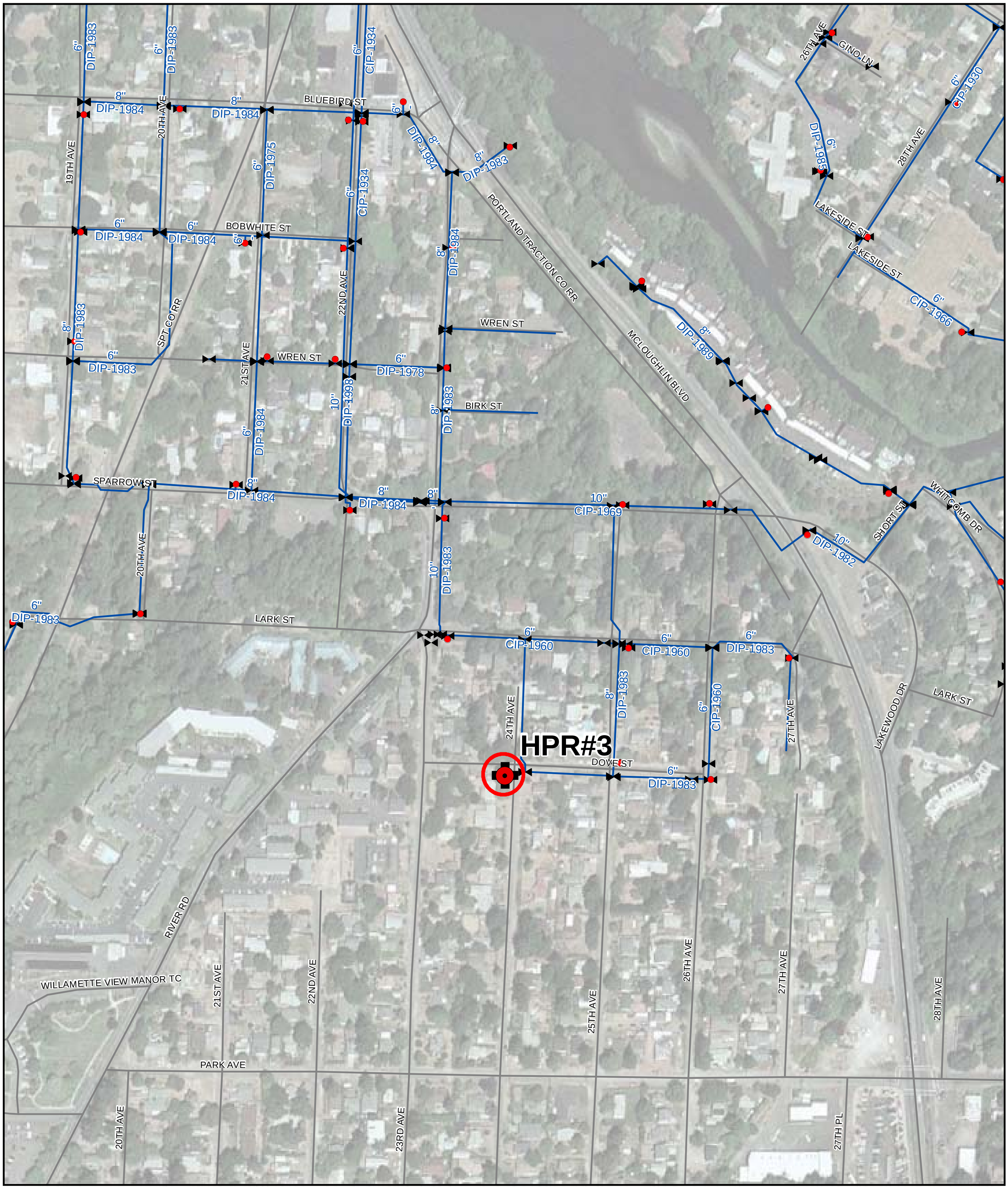
O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\HPR\Fig_HPR#1.mxd 6/28/2010

FIGURE B-2

City of Milwaukie
Water System Master Plan

HPR #2

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\HPR\Fig_HPR#2.mxd 6/28/2010



LEGEND

- HPR Location
- Hydrant
- Valve
- Pipeline

0 190 380

Scale in Feet

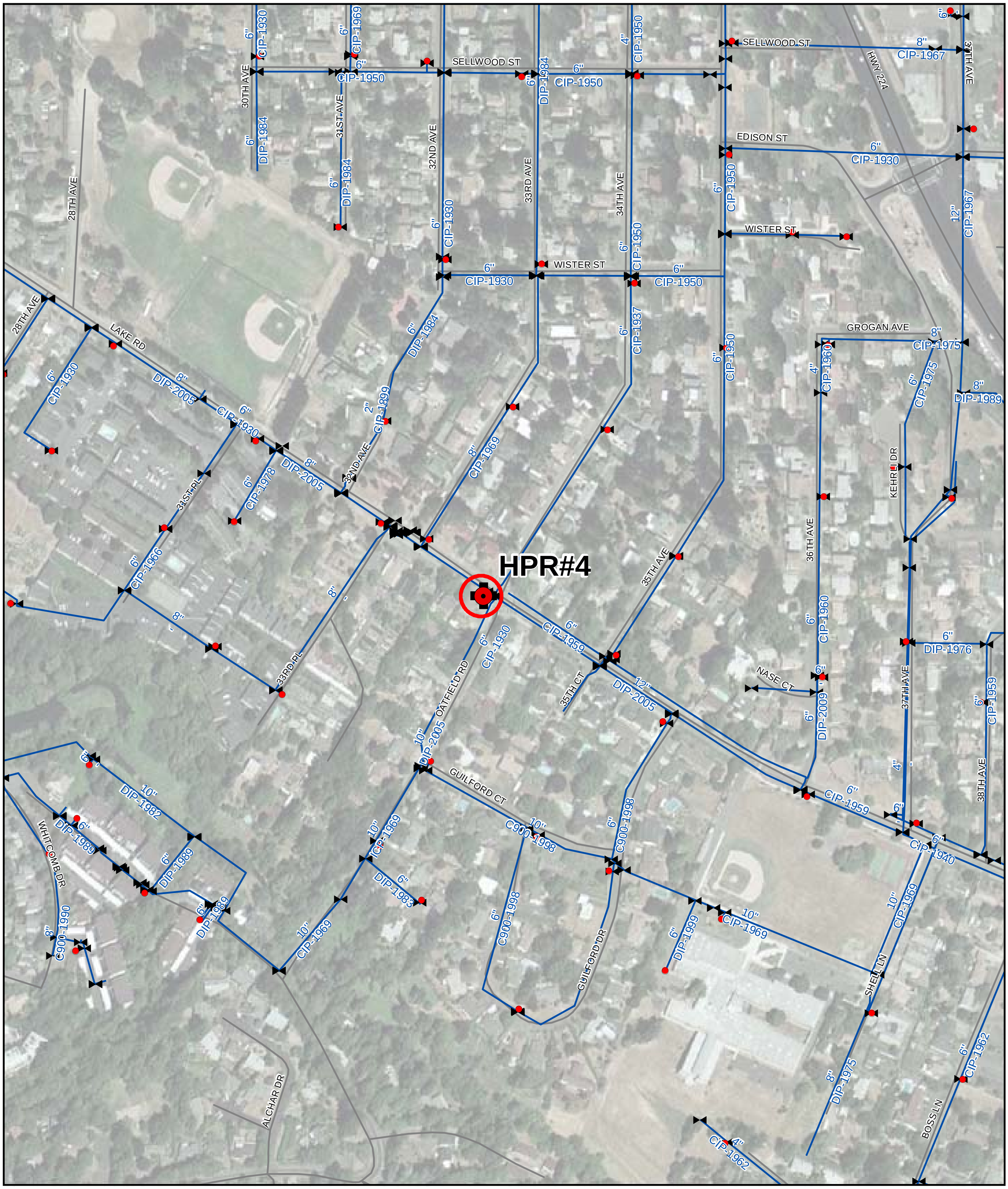
FIGURE B-3

City of Milwaukie
Water System Master Plan

HPR #3

WEST YOST ASSOCIATES
Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\HPR\Fig_HPR#3.mxd 6/28/2010



LEGEND

- HPR Location
- Hydrant
- Valve
- Pipeline

0 190 380

Scale in Feet

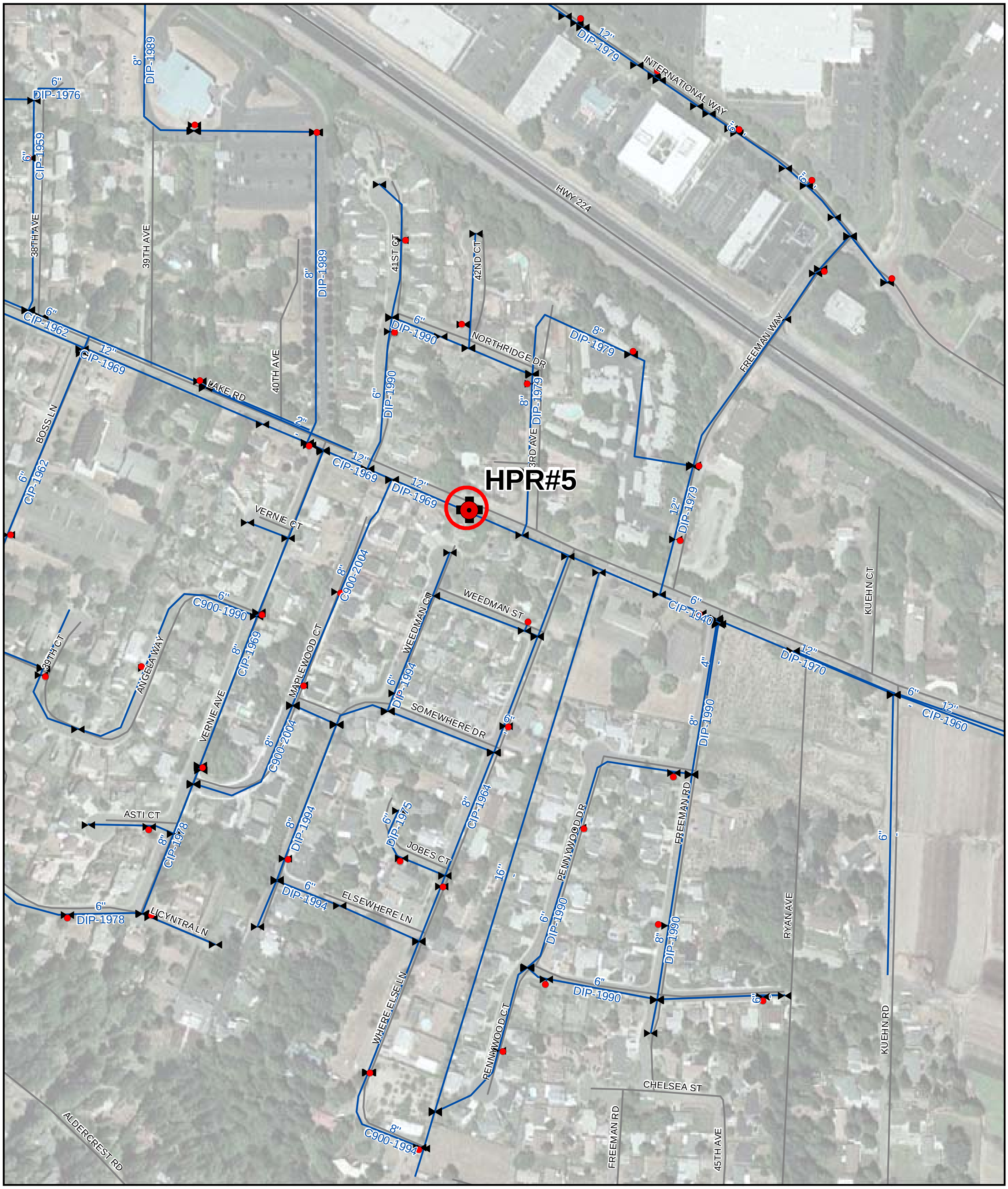
FIGURE B-4

City of Milwaukie
Water System Master Plan

HPR #4

WEST YOST ASSOCIATES
Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\HPR\Fig_HPR#4.mxd 6/28/2010



LEGEND

- HPR Location
- Hydrant
- Valve
- Pipeline

0 190 380

Scale in Feet

FIGURE B-5

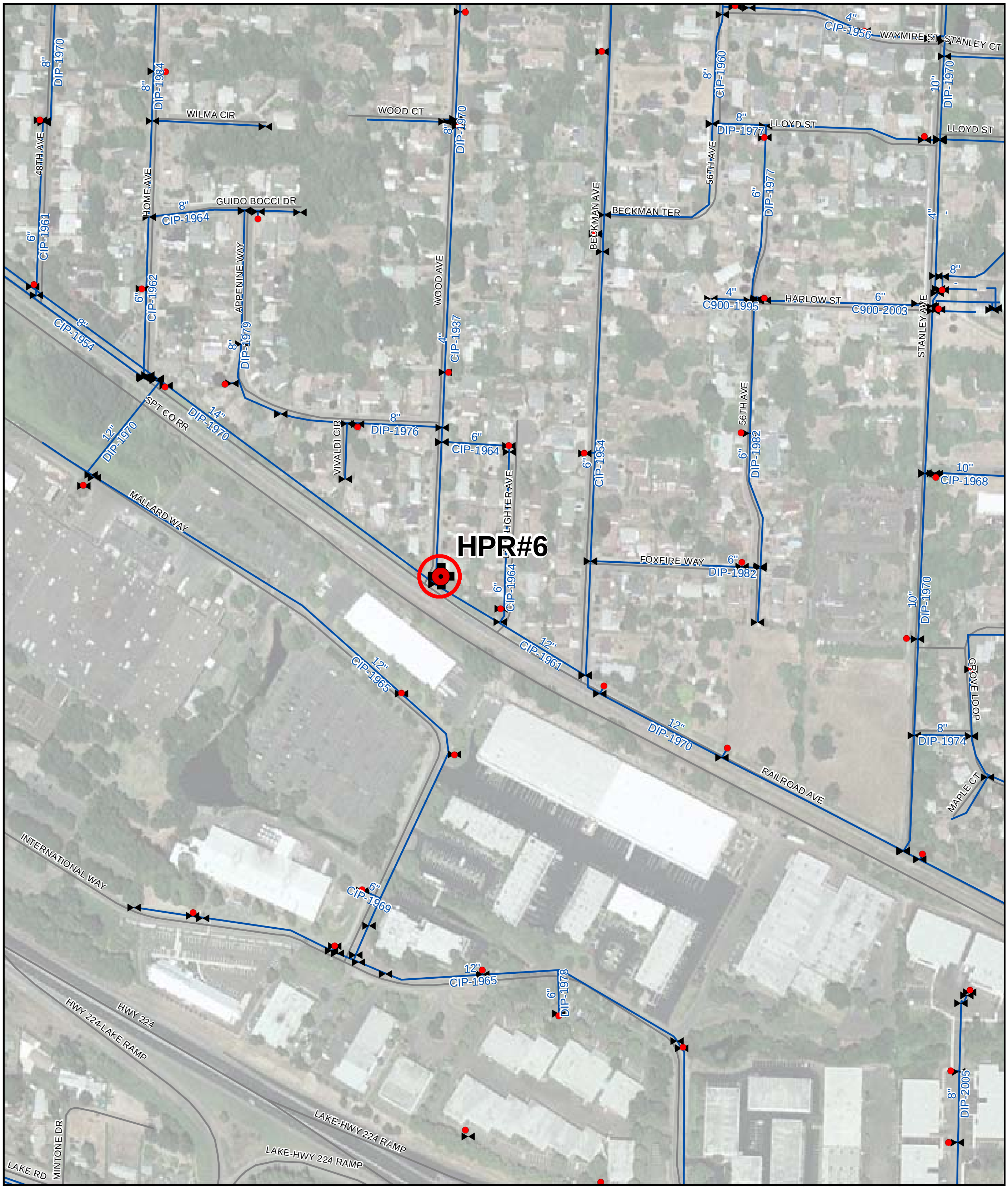
City of Milwaukie
Water System Master Plan

HPR #5

WEST YOST ASSOCIATES

Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\HPR\Fig_HPR#5.mxd 7/7/2010



LEGEND

- HPR Location
- Hydrant
- Valve
- Pipeline

0 190 380

Scale in Feet

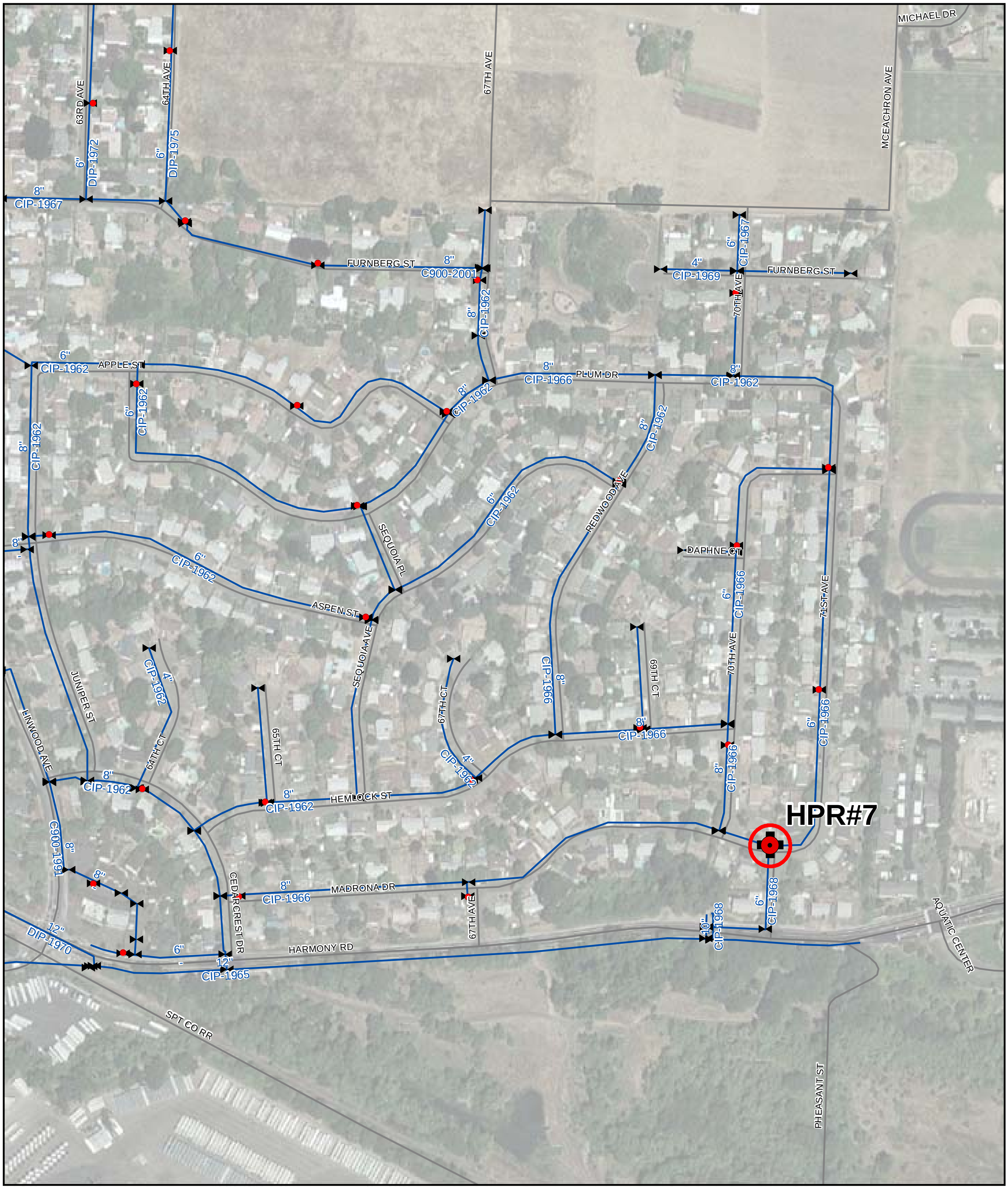
FIGURE B-6

City of Milwaukie Water System Master Plan

HPR #6

WEST YOST ASSOCIATES Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\HPR\Fig_HPR#6.mxd 6/28/2010



LEGEND

- HPR Location
- Hydrant
- Valve
- Pipeline

0 190 380

Scale in Feet

FIGURE B-7

City of Milwaukie
Water System Master Plan

HPR #7

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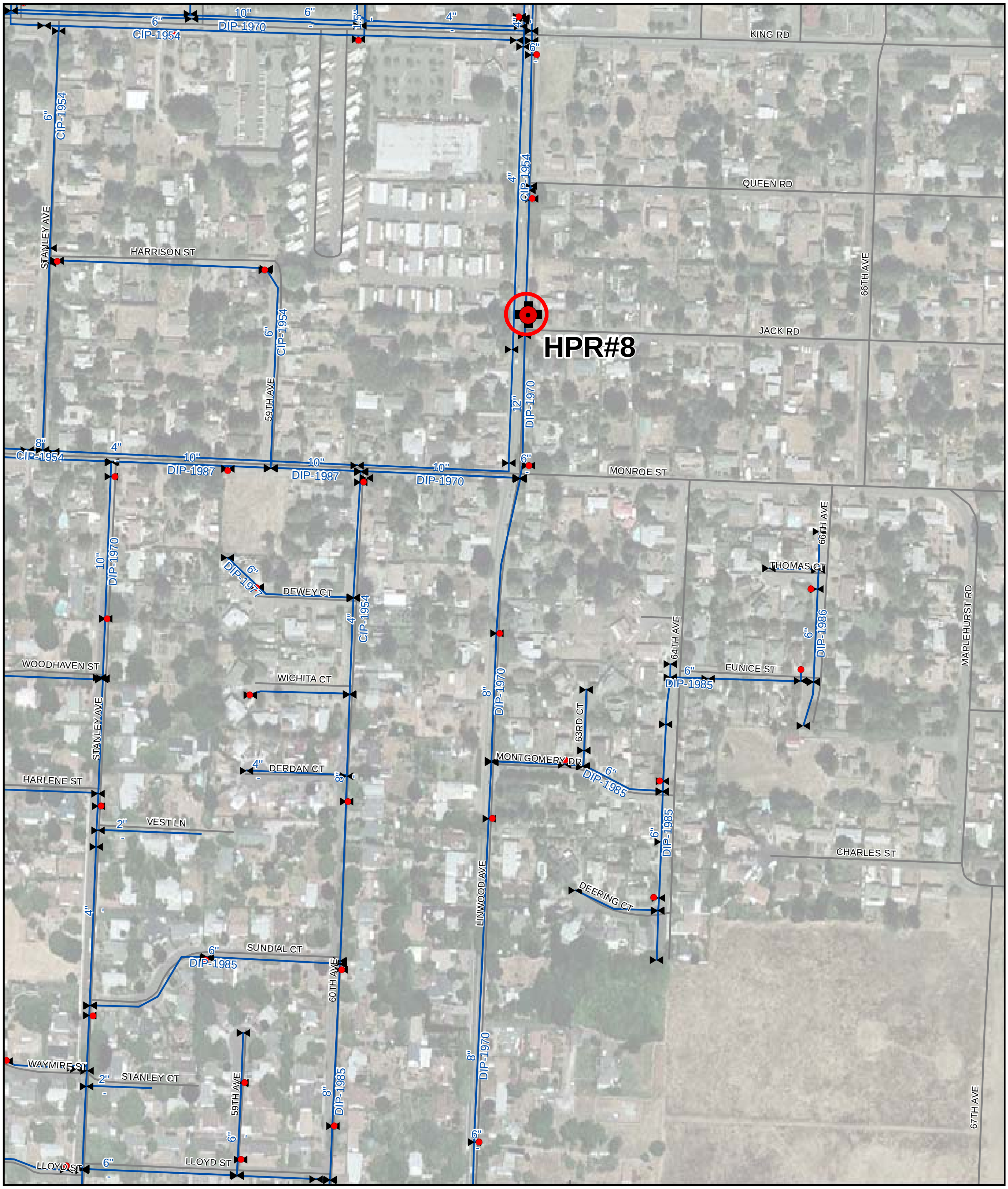
Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\HPR\Fig_HPR#7.mxd 6/28/2010



LEGEND

HPR Location

Hydrant

Valve

Pipeline

N

0 190 380

Scale in Feet

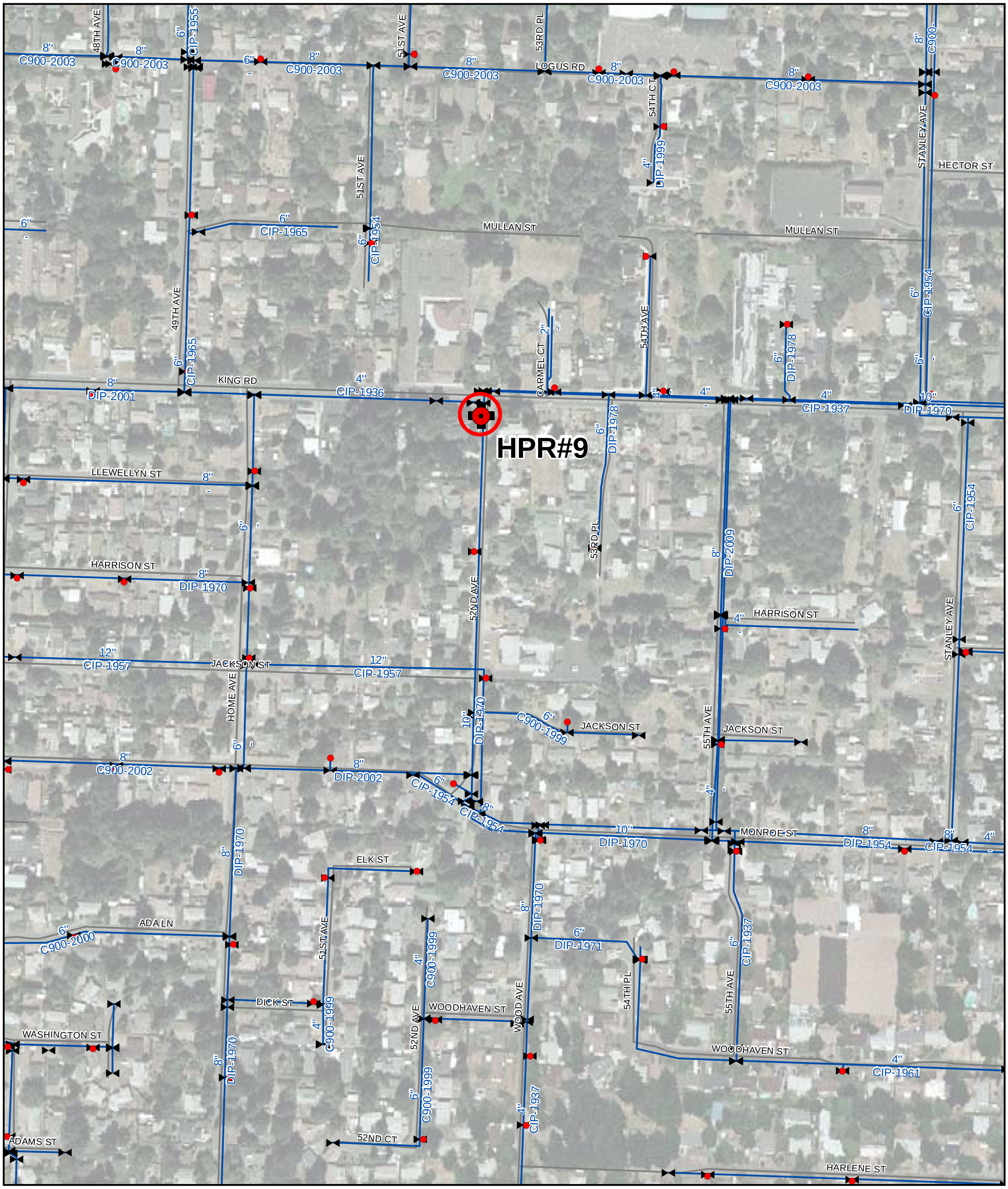
FIGURE B-8

City of Milwaukie
Water System Master Plan

HPR #8

WEST YOST ASSOCIATES
Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\HPR\Fig_HPR#8.mxd 6/28/2010



LEGEND

- HPR Location
- Hydrant
- Valve
- Pipeline

N

0 190 380

Scale in Feet

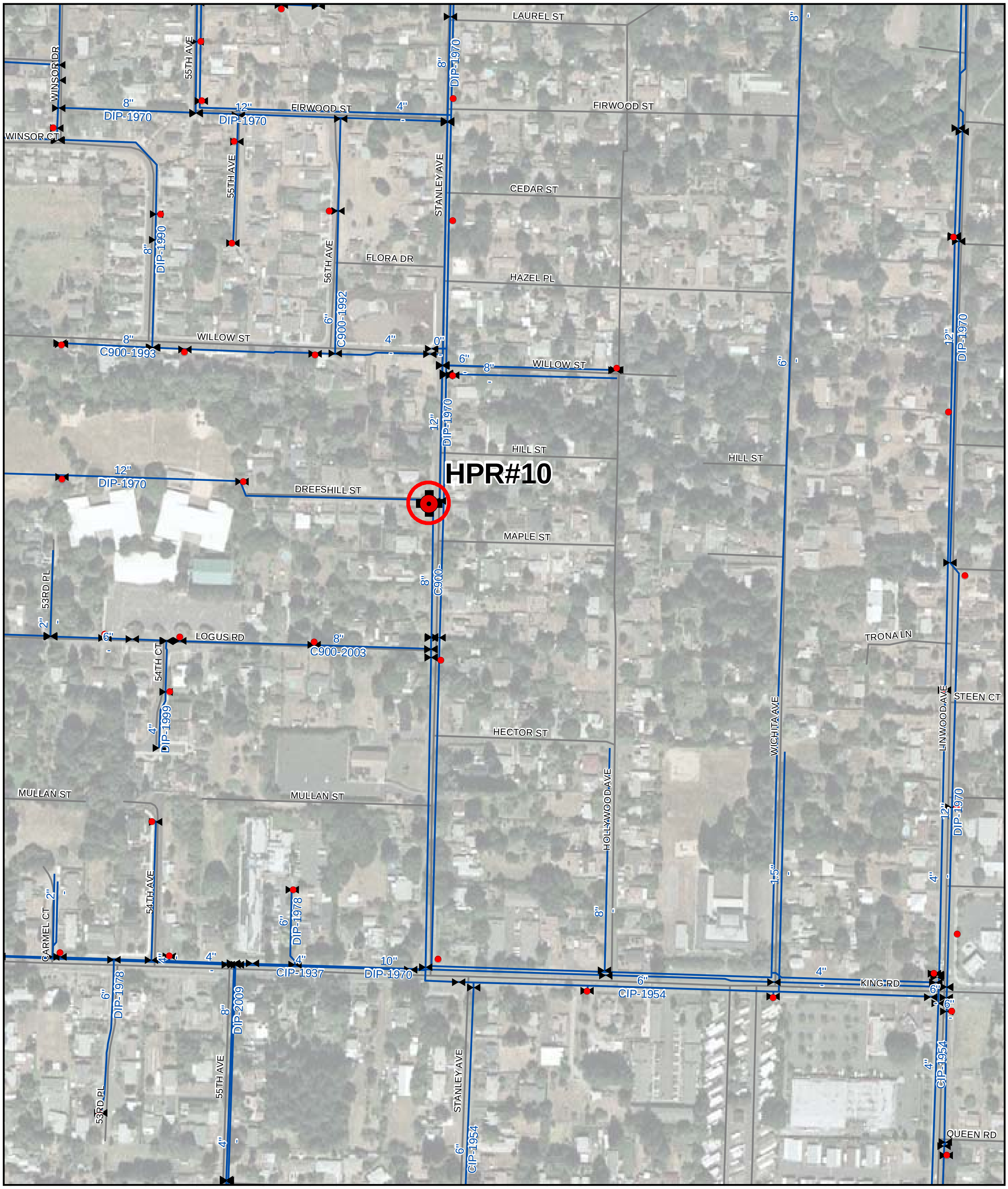
FIGURE B-9

City of Milwaukie
Water System Master Plan

HPR #9

WEST YOST ASSOCIATES
Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\HPR\Fig_HPR#9.mxd 6/28/2010



LEGEND

- HPR Location
- Hydrant
- Valve
- Pipeline

0 190 380

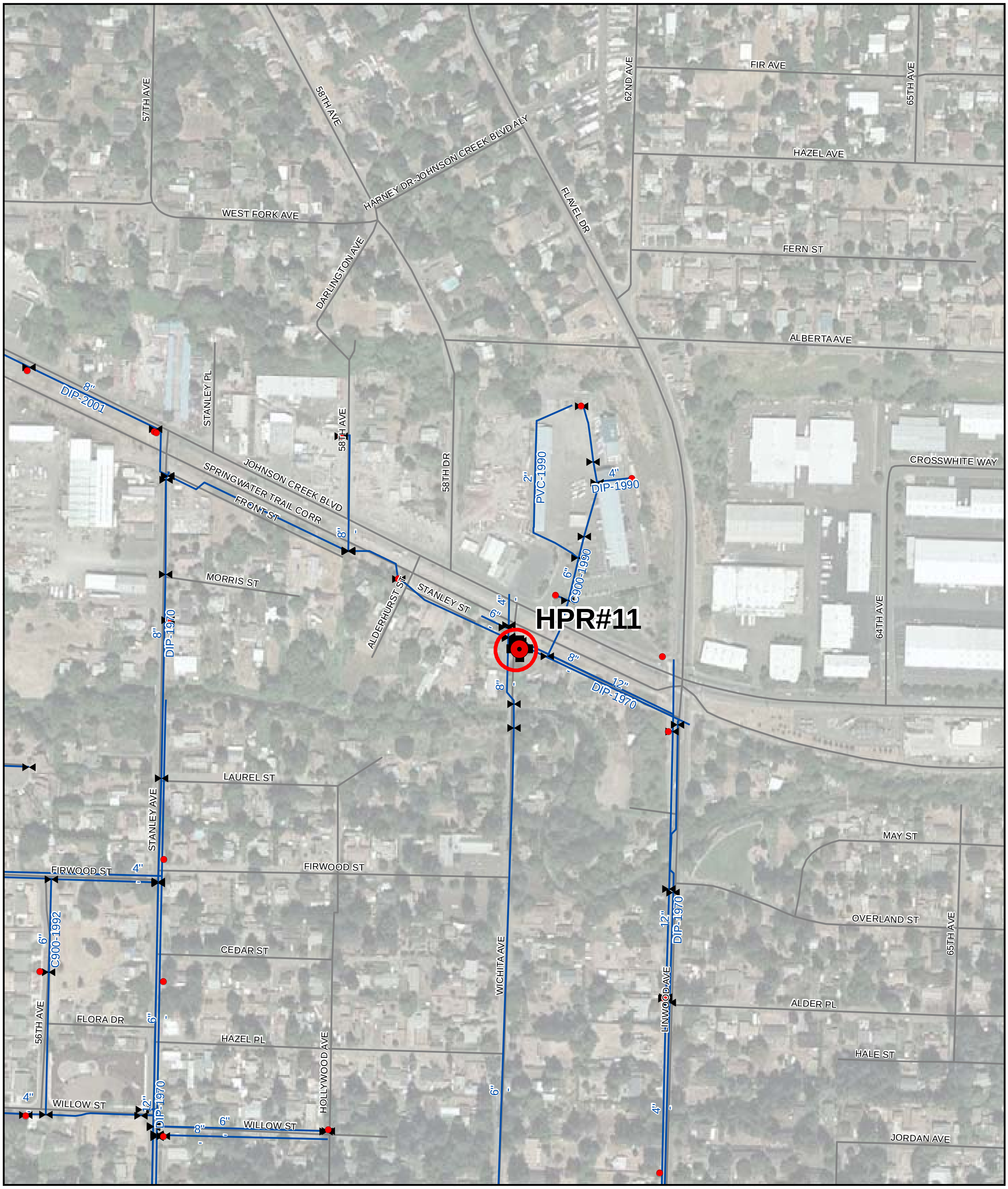
Scale in Feet

FIGURE B-10

City of Milwaukie
Water System Master Plan

HPR #10

WEST YOST ASSOCIATES
Consulting Engineers



LEGEND

- HPR Location
- Hydrant
- Valve
- Pipeline

0 190 380

Scale in Feet

N

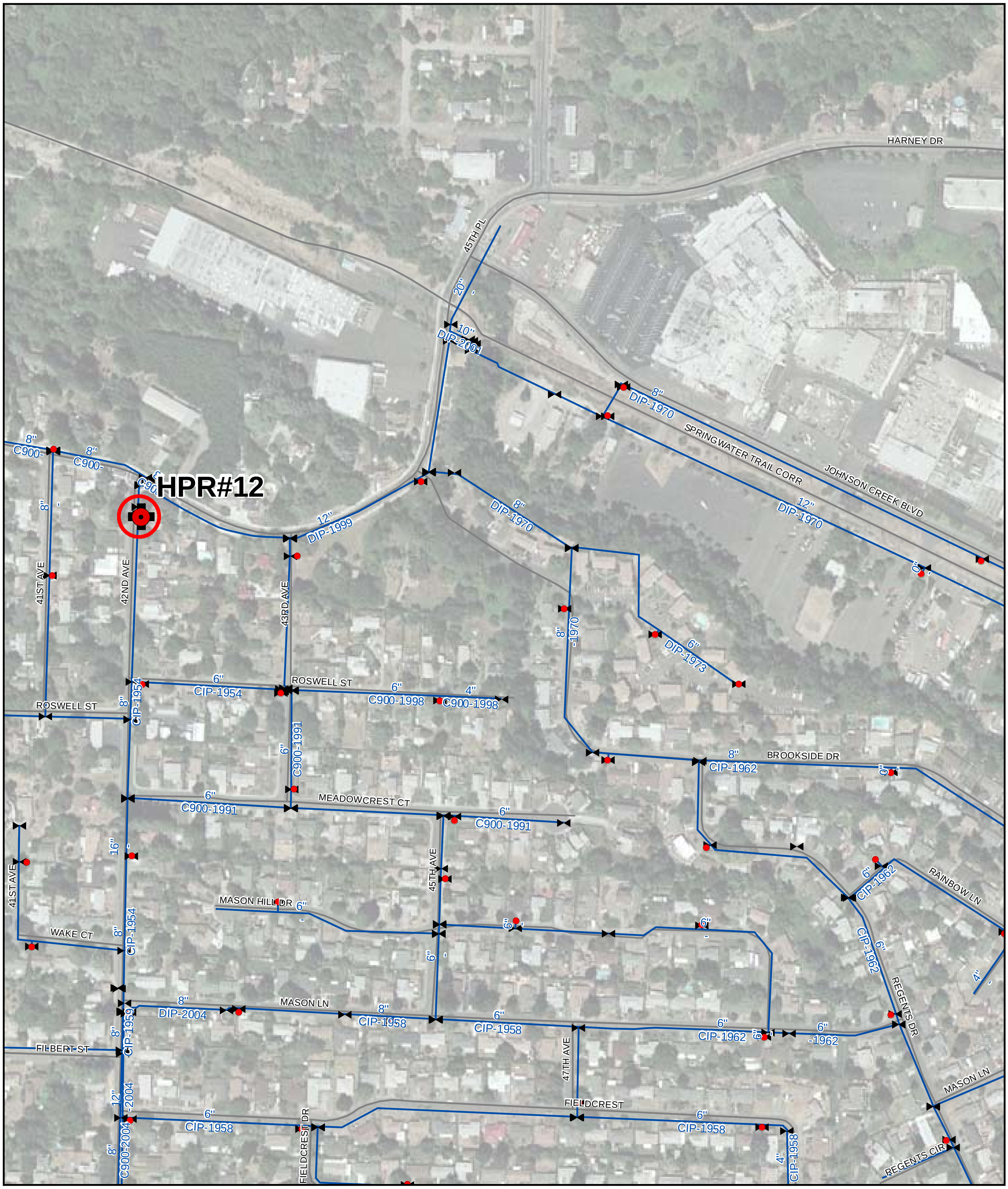
FIGURE B-11

City of Milwaukie Water System Master Plan

HPR #11

WEST YOST ASSOCIATES Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\HPR\Fig_HPR#11.mxd 6/29/2010



LEGEND

- HPR Location
- Hydrant
- Valve
- Pipeline

N

0 190 380

Scale in Feet

FIGURE B-12

City of Milwaukie
Water System Master Plan

HPR #12

WEST YOST ASSOCIATES
Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\HPR\Fig_HPR#12.mxd 6/28/2010

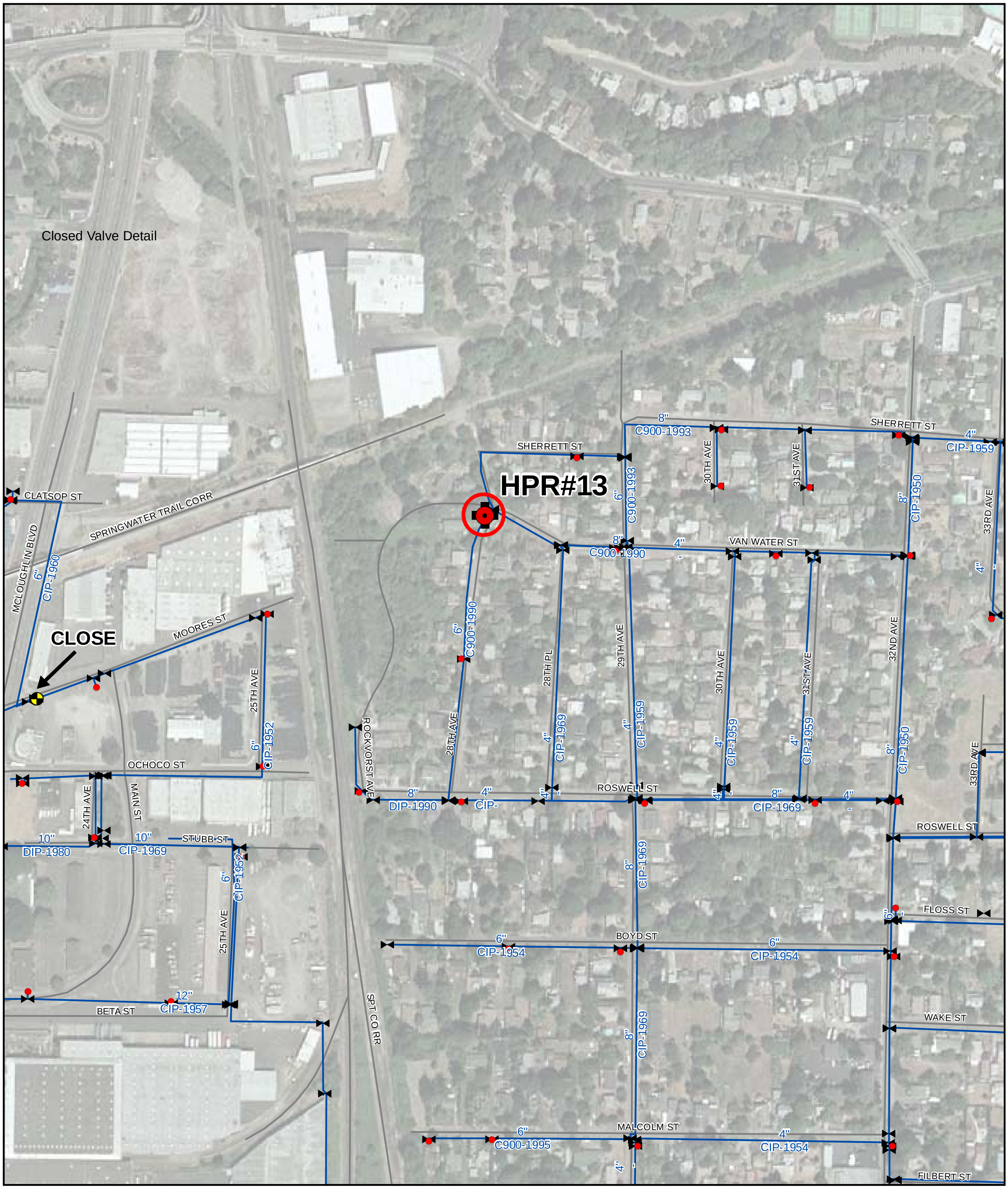


FIGURE B-13

LEGEND

- HPR Location
- Hydrant
- Valve
- Pipeline

0 190 380

Scale in Feet

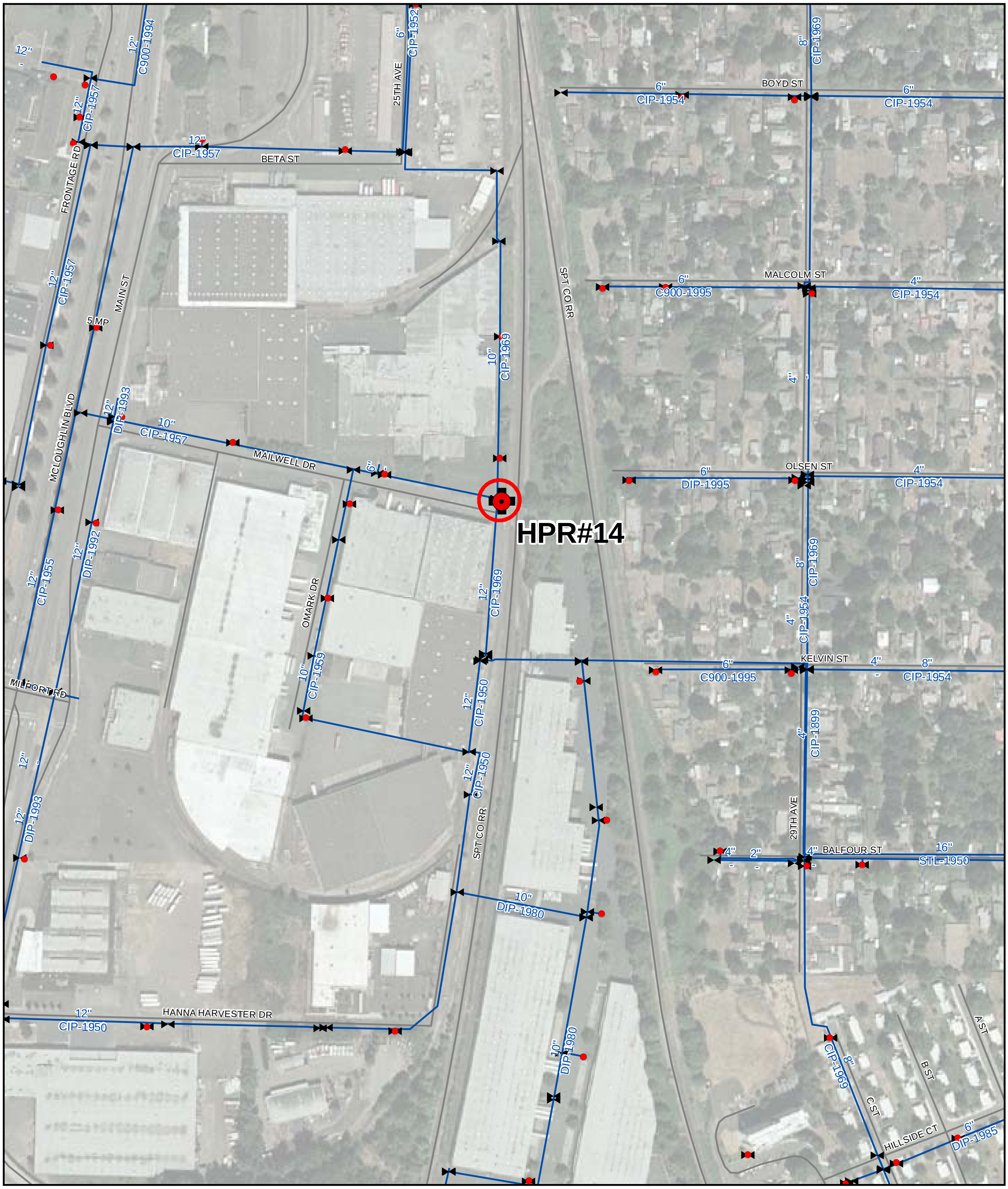
N

City of Milwaukie
Water System Master Plan

HPR #13

WEST YOST ASSOCIATES
Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\HPR\Fig_HPR#13.mxd 6/28/2010



LEGEND

- HPR Location
- Hydrant
- Valve
- Pipeline

0 190 380

Scale in Feet

FIGURE B-14

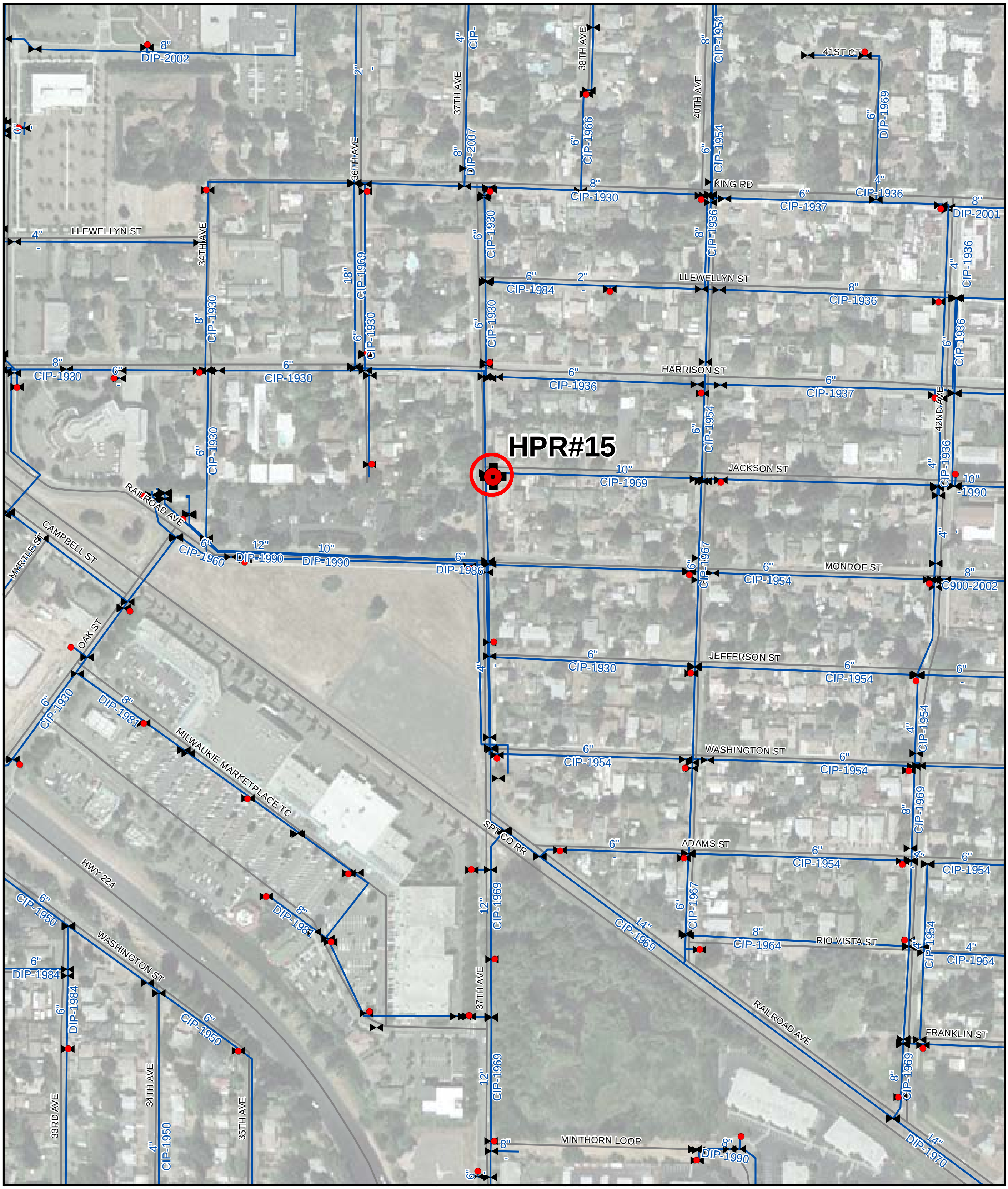
City of Milwaukie Water System Master Plan

HPR #14

WEST YOST ASSOCIATES

Consulting Engineers

O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\HPR\Fig_HPR#14.mxd 6/28/2010



LEGEND

HPR Location

Hydrant

Valve

Pipeline

0 190 380

Scale in Feet

N

FIGURE B-15

City of Milwaukie
Water System Master Plan

HPR #15

WEST YOST ASSOCIATES
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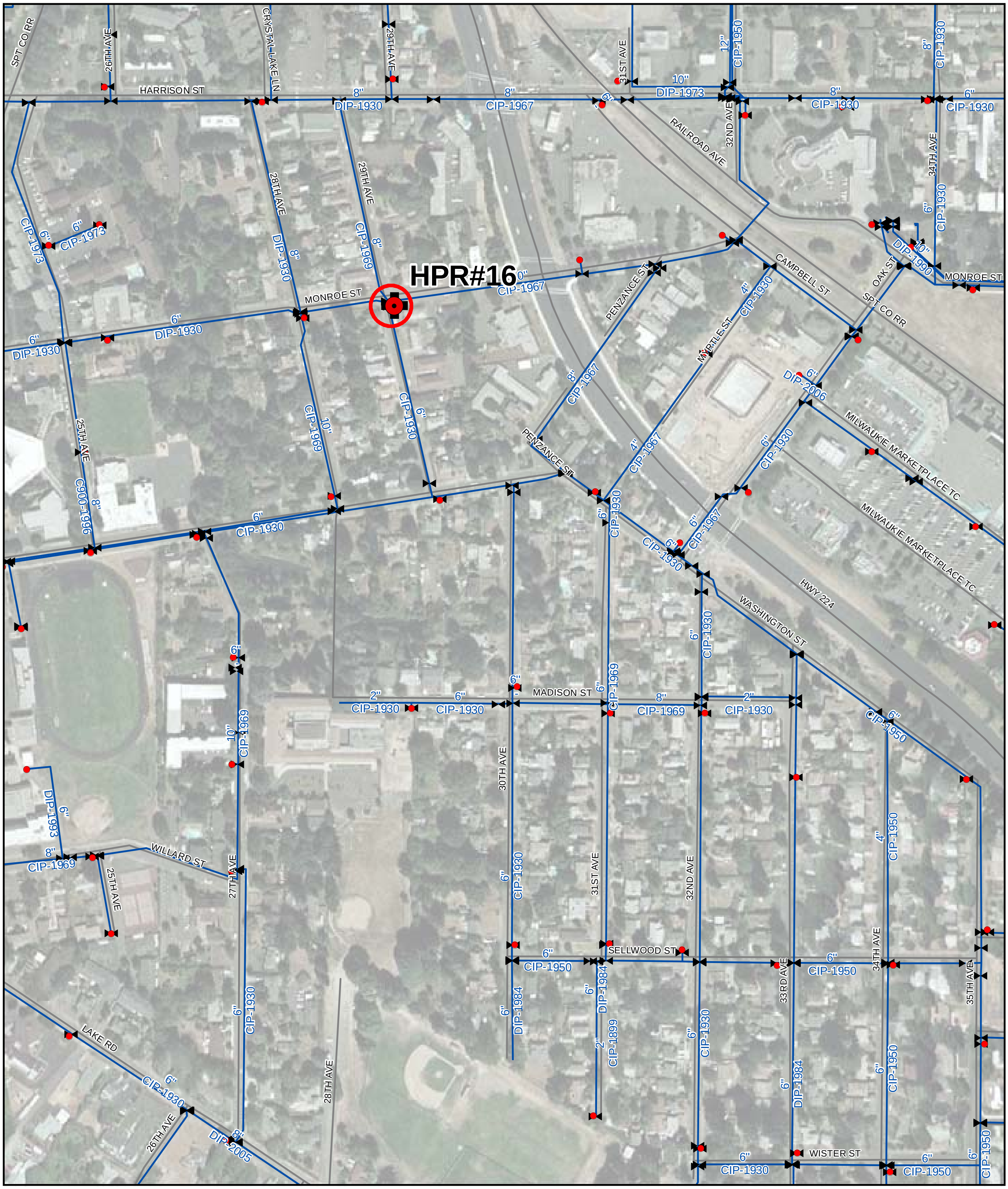
O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\HPR\Fig_HPR#15.mxd 6/28/2010



LEGEND

- HPR Location
- Hydrant
- Valve
- Pipeline

0 190 380

Scale in Feet

FIGURE B-16

City of Milwaukie Water System Master Plan

HPR #16

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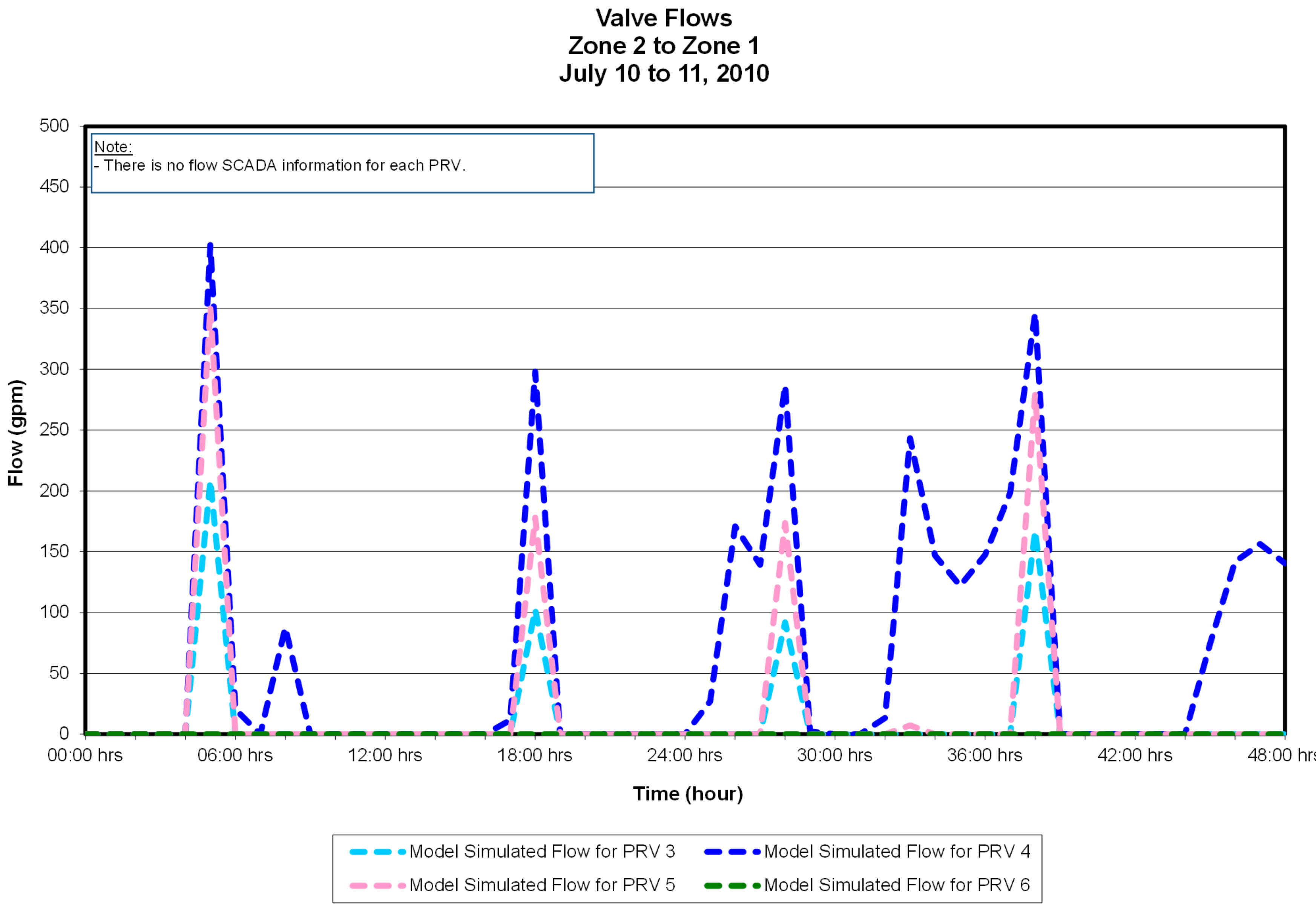
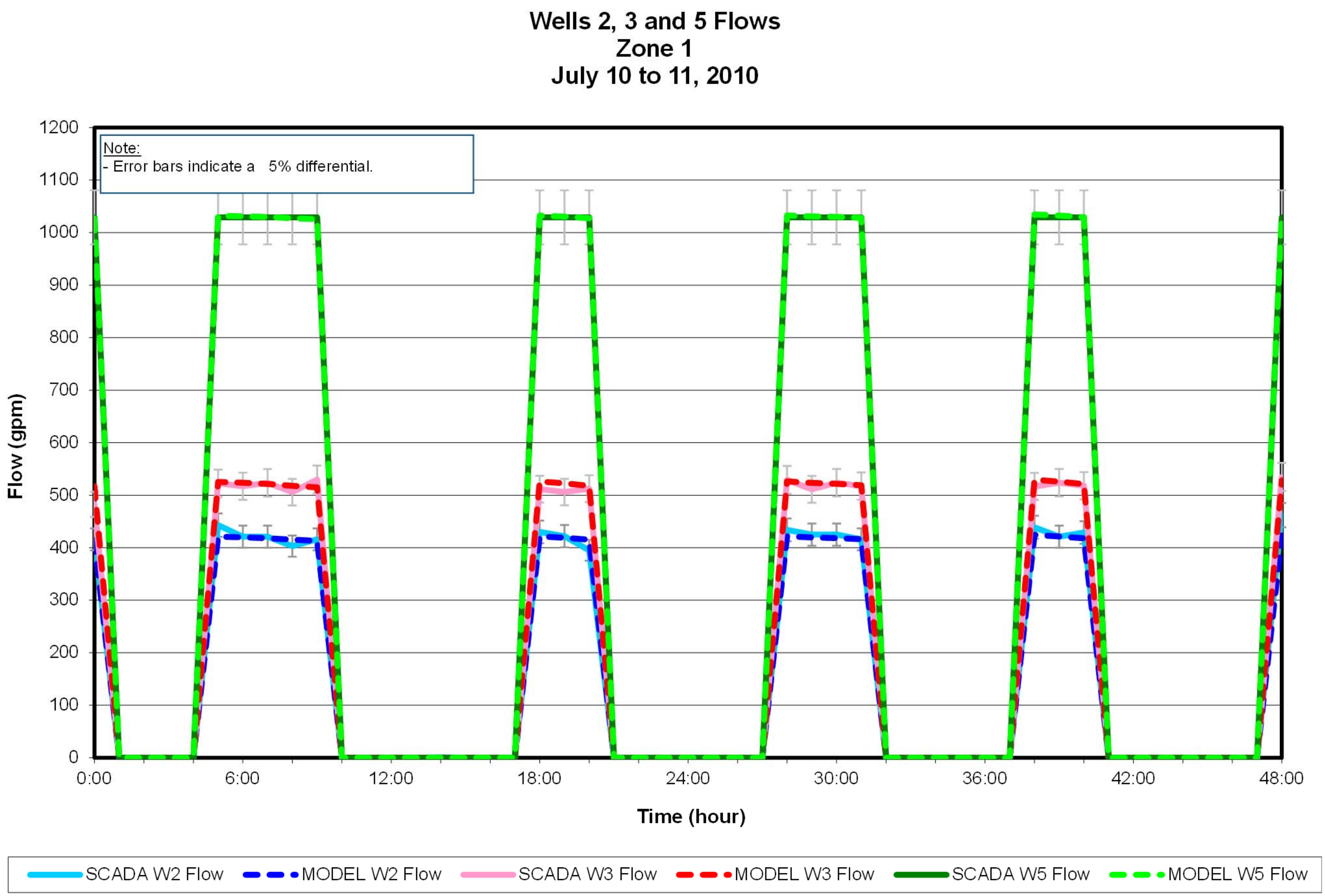
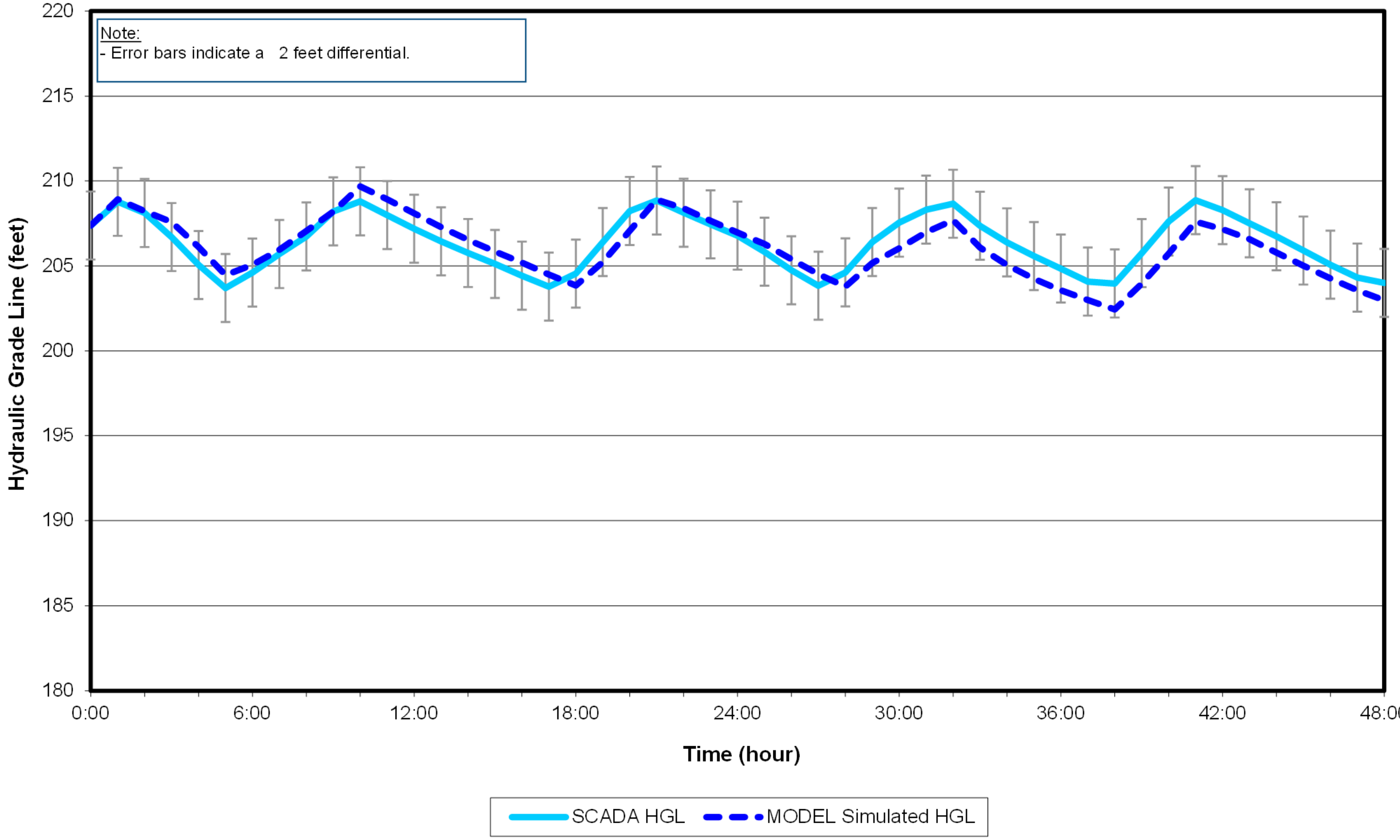
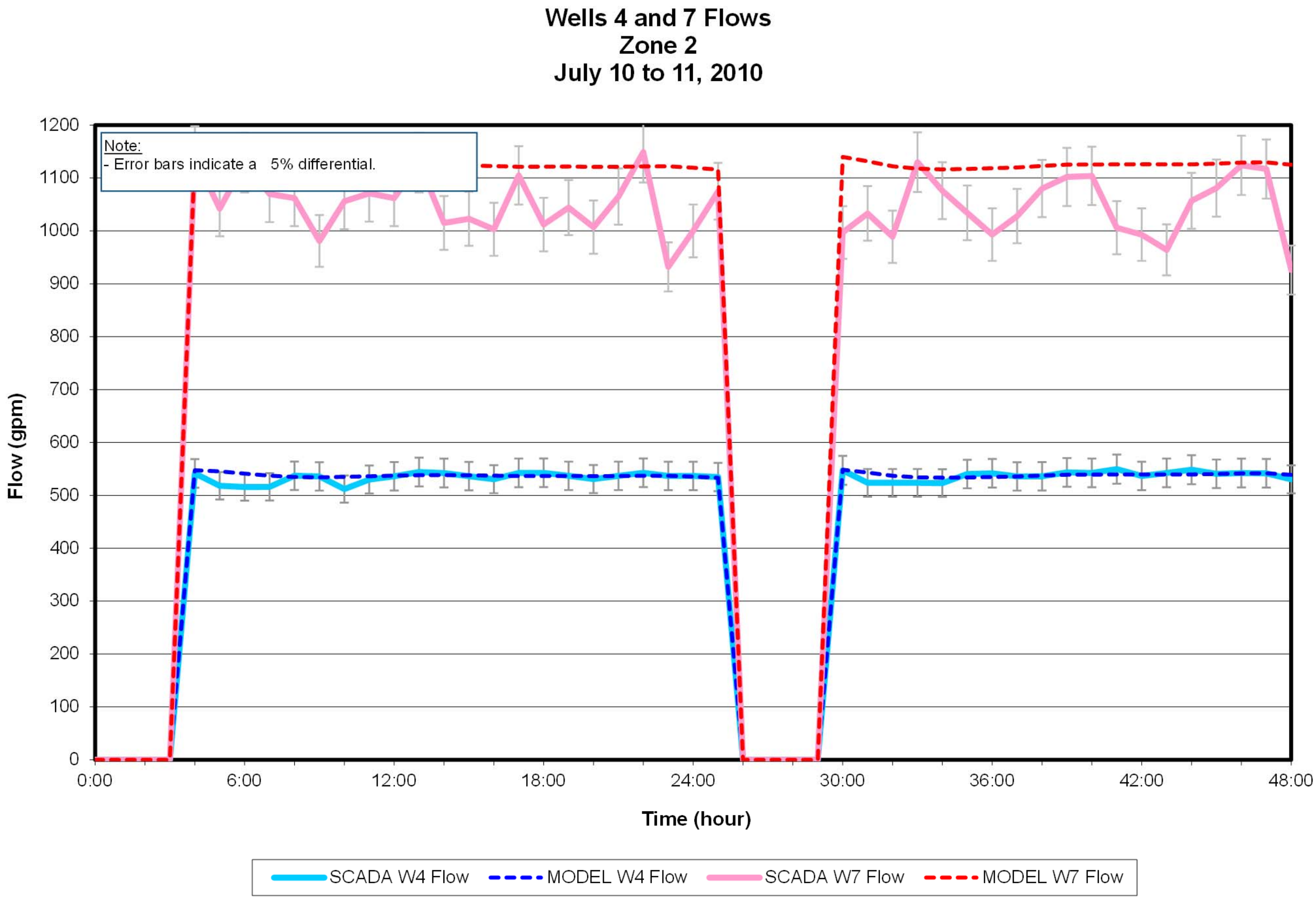
O:\Clients\382 City of Milwaukie\03-10-01\GIS\Figures\HPR\Fig_HPR#16.mxd 6/28/2010


Valve Flows
Zone 2 to Zone 1
July 10 to 11, 2010
Note:
- There is no flow SCADA information for each PRV.
Flow (gpm)
500
450
400
350
300
250
200
150
100
50
0
00:00 hrs
06:00 hrs
12:00 hrs
18:00 hrs
24:00 hrs
30:00 hrs
36:00 hrs
42:00 hrs
48:00 hrs
Time (hour)
Model Simulated Flow for PRV 3
Model Simulated Flow for PRV 4
Model Simulated Flow for PRV 5
Model Simulated Flow for PRV 6


Wells 2, 3 and 5 Flows
Zone 1
July 10 to 11, 2010
Note:
- Error bars indicate a 5% differential.
Flow (gpm)
1200
1100
1000
900
800
700
600
500
400
300
200
100
0
0:00
6:00
12:00
18:00
24:00
30:00
36:00
42:00
48:00
Time (hour)
SCADA W2 Flow
MODEL W2 Flow
SCADA W3 Flow
MODEL W3 Flow
SCADA W5 Flow
MODEL W5 Flow

Concrete Tank Level
Zone 1
July 10 to 11, 2010





W E S T Y O S T A S S O C I A T E S
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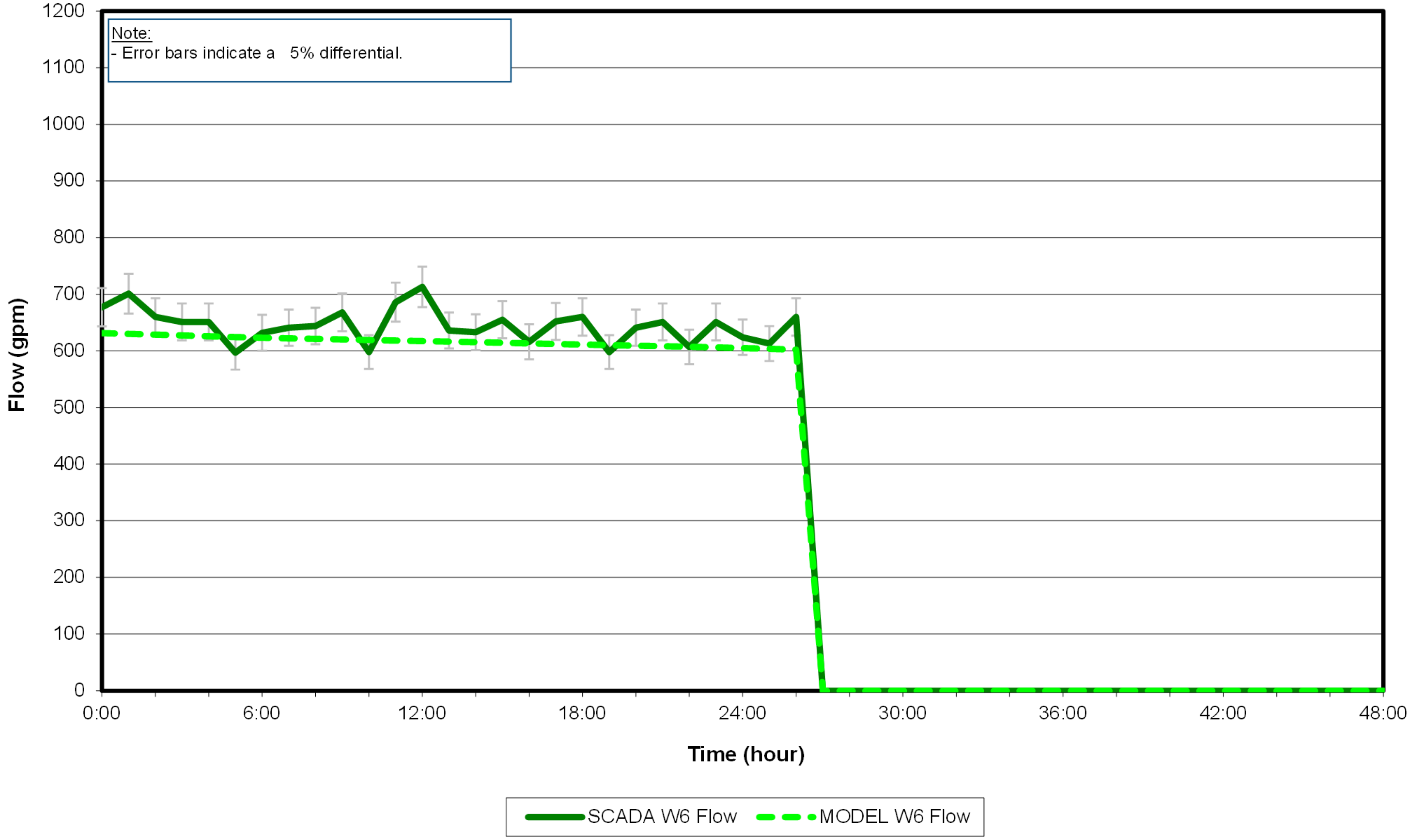
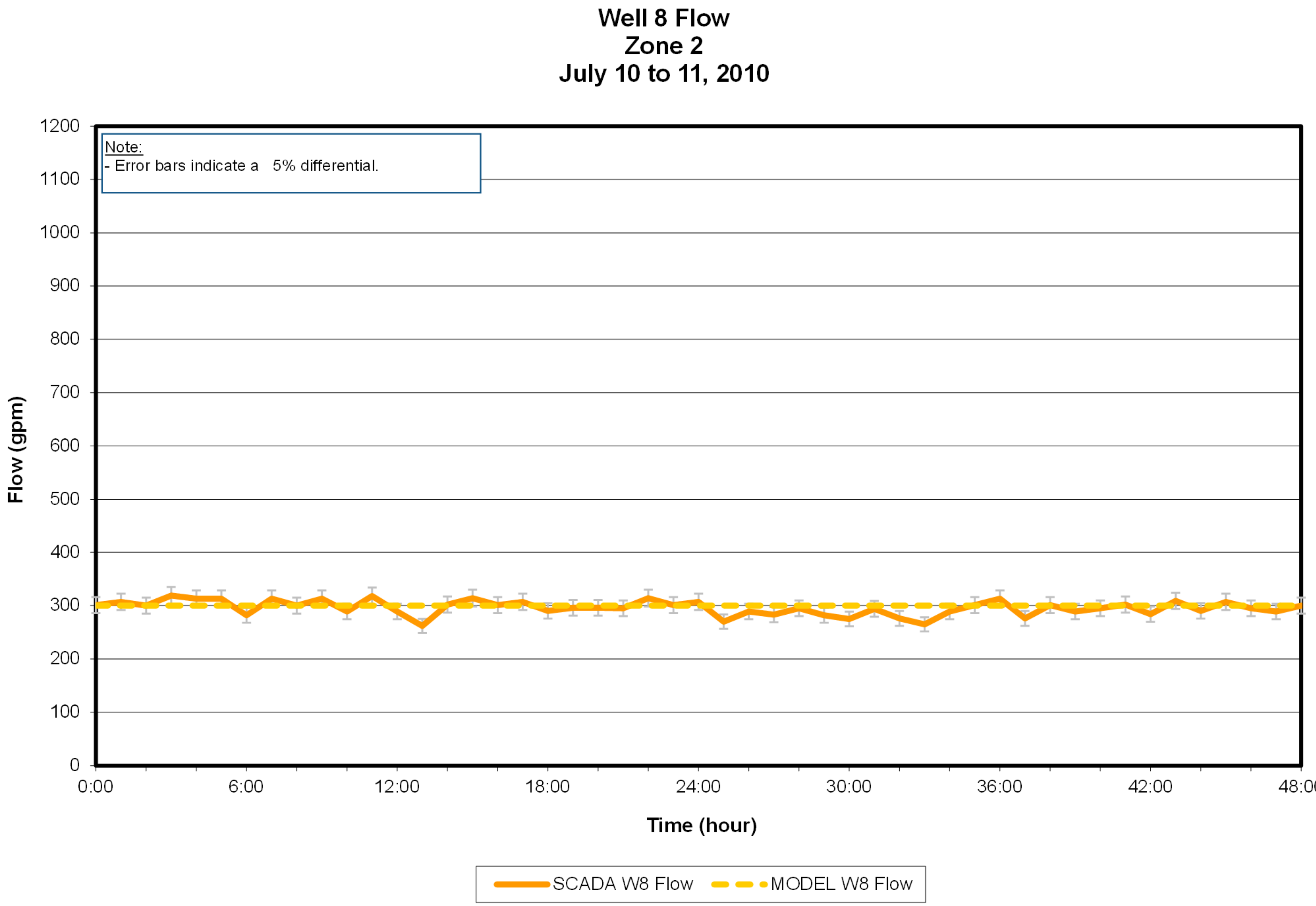
Last Revised: 11-17-10

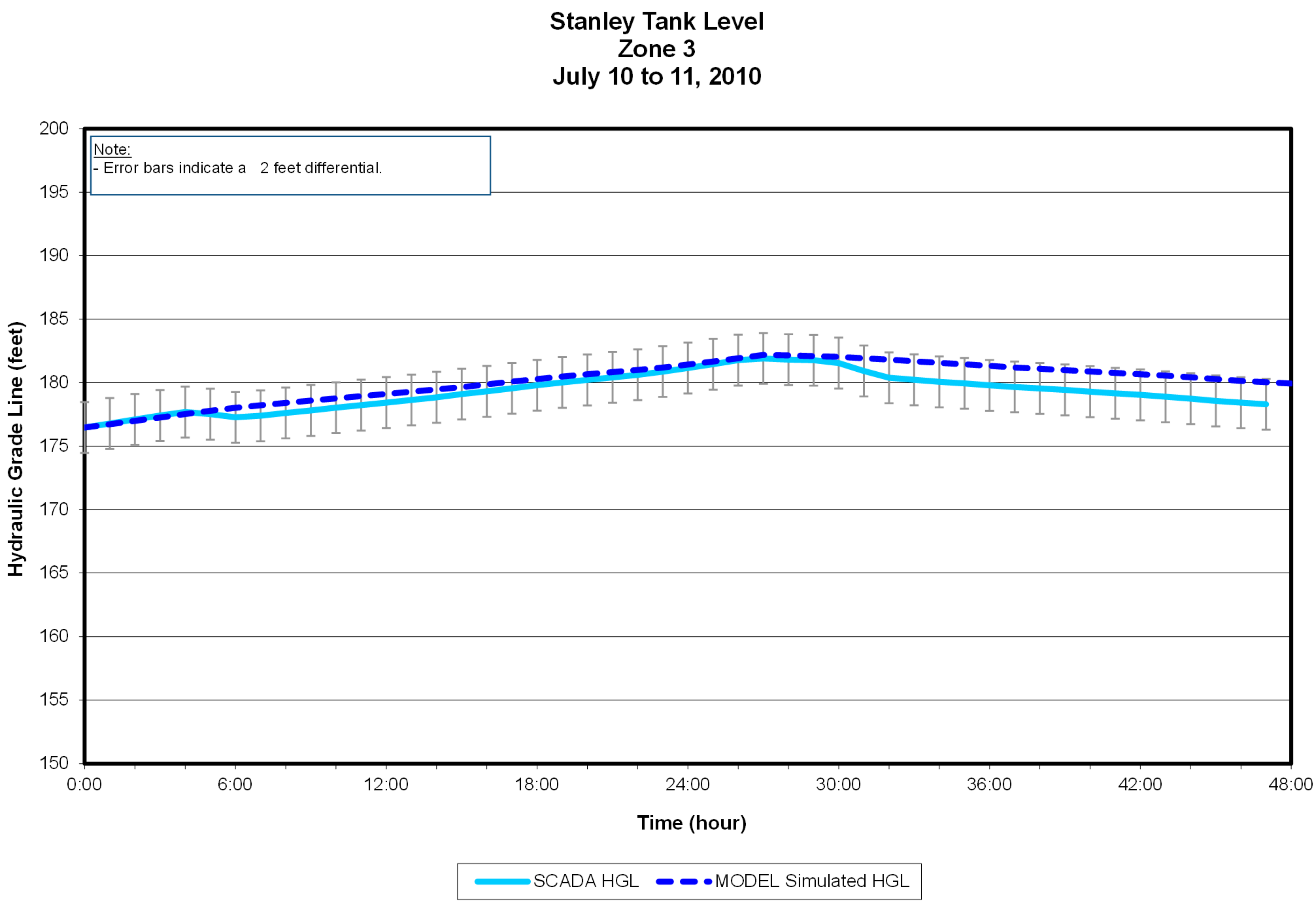
City of Milwaukie
Water Master Plan

Well 6 Flow
Zone 2
July 10 to 11, 2010

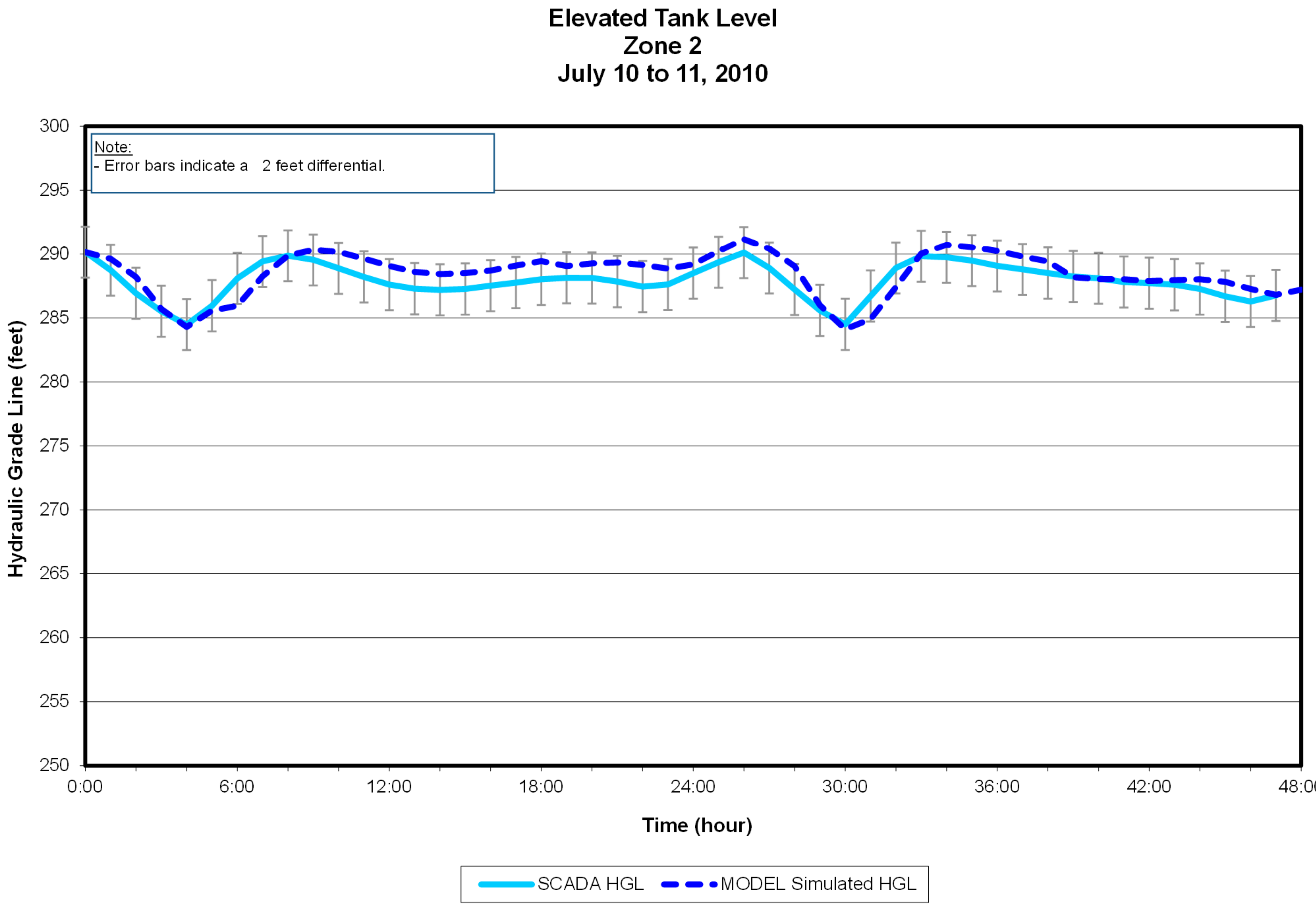


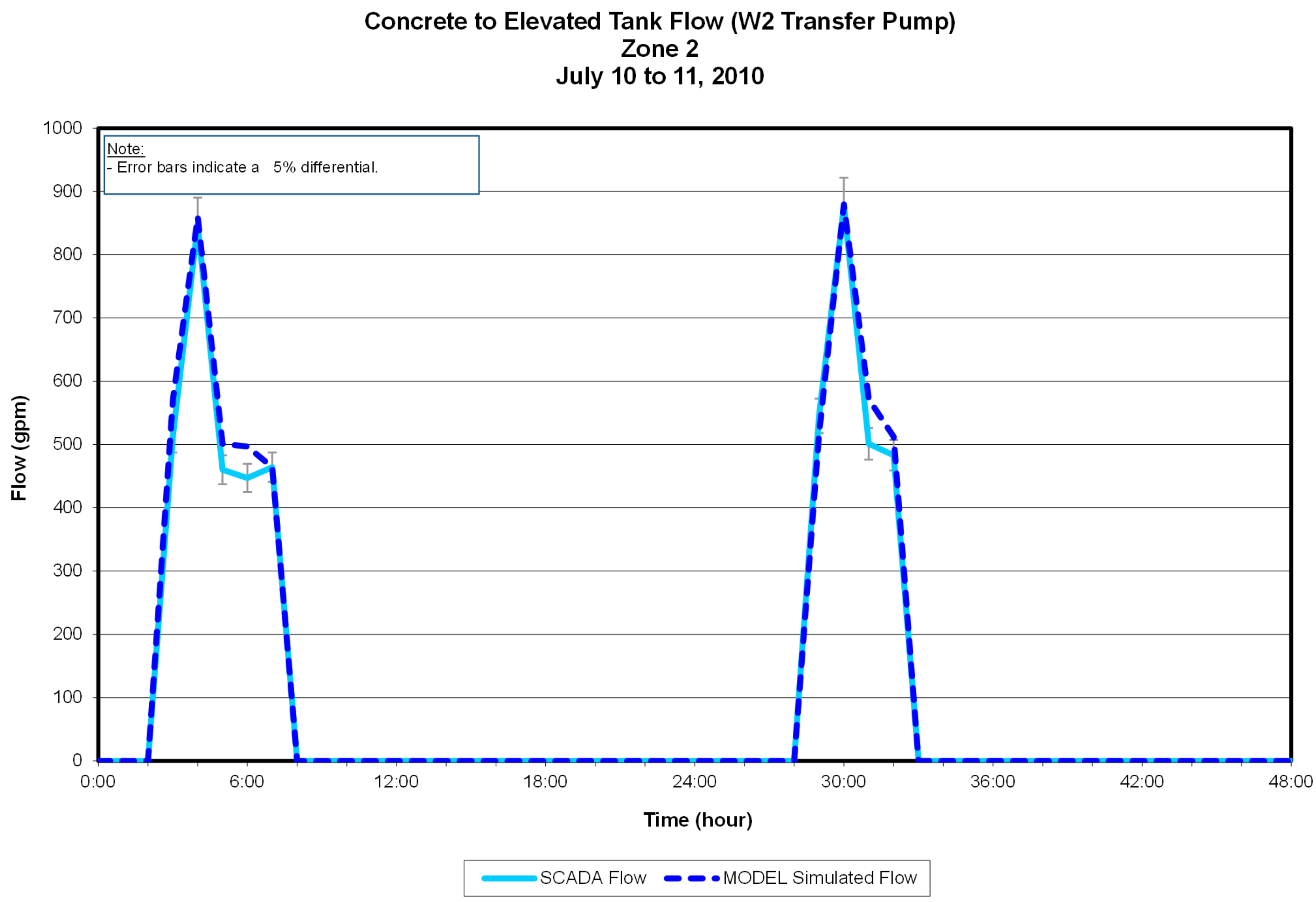


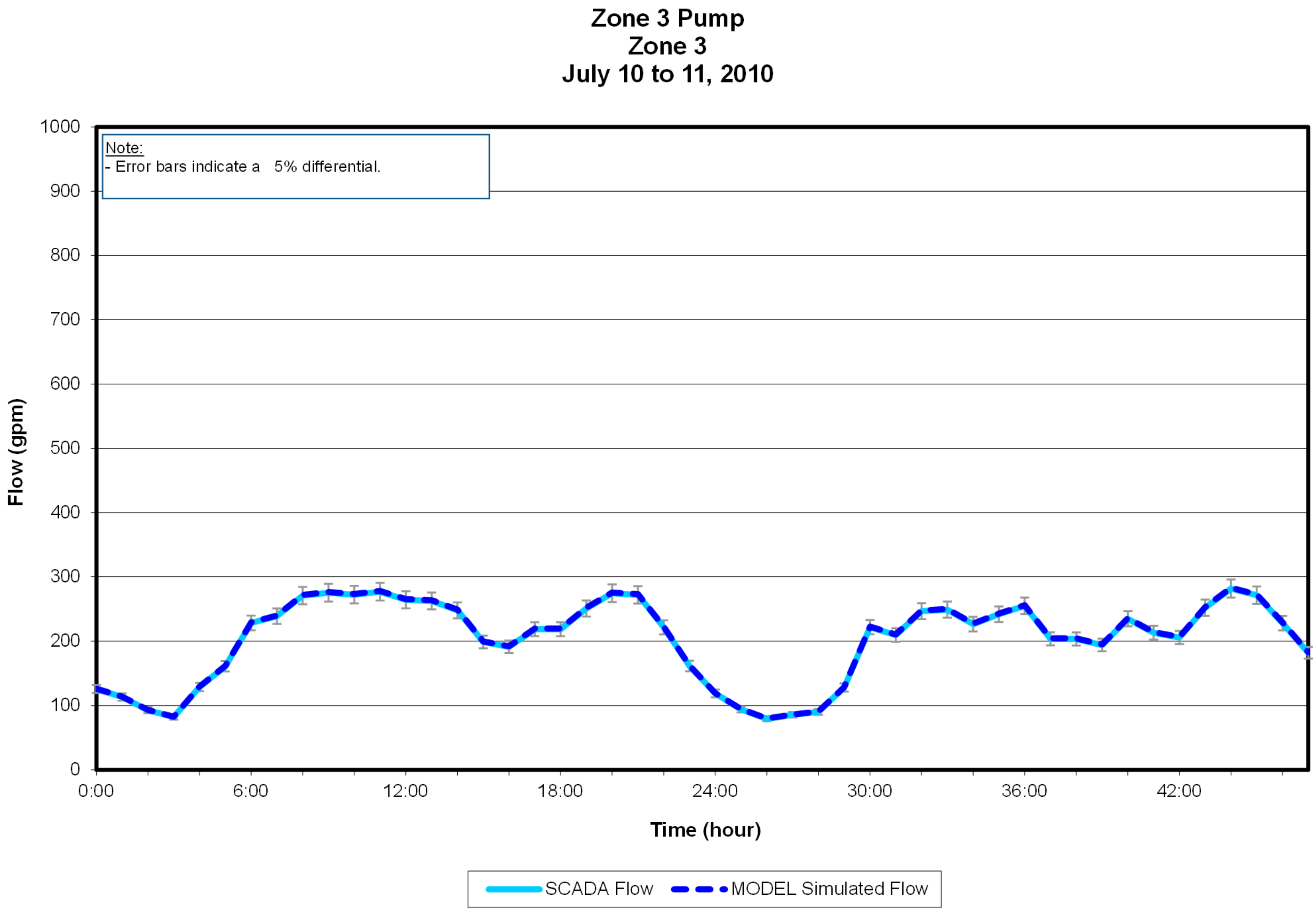
Well 8 Flow
Zone 2
July 10 to 11, 2010
Note:
- Error bars indicate a 5% differential.
Flow (gpm)
1200
1100
1000
900
800
700
600
500
400
300
200
100
0
0:00
6:00
12:00
18:00
24:00
30:00
36:00
42:00
48:00
Time (hour)
SCADA W8 Flow
MODEL W8 Flow



Stanley Tank Level
Zone 3
July 10 to 11, 2010
Note:
- Error bars indicate a 2 feet differential.
Hydraulic Grade Line (feet)
200
195
190
185
180
175
170
165
160
155
150
0:00
6:00
12:00
18:00
24:00
30:00
36:00
42:00
48:00
Time (hour)
SCADA HGL
MODEL Simulated HGL


Elevated Tank Level
Zone 2
July 10 to 11, 2010
Note:
- Error bars indicate a 2 feet differential.
Hydraulic Grade Line (feet)
300
295
290
285
280
275
270
265
260
255
250
0:00
6:00
12:00
18:00
24:00
30:00
36:00
42:00
48:00
Time (hour)
SCADA HGL
MODEL Simulated HGL


Concrete to Elevated Tank Flow (W2 Transfer Pump)
Zone 2
July 10 to 11, 2010
Note:
- Error bars indicate a 5% differential.
1000
900
800
700
600
500
400
300
200
100
0
Flow (gpm)
0:00
6:00
12:00
18:00
24:00
30:00
36:00
42:00
48:00
Time (hour)
SCADA Flow
MODEL Simulated Flow



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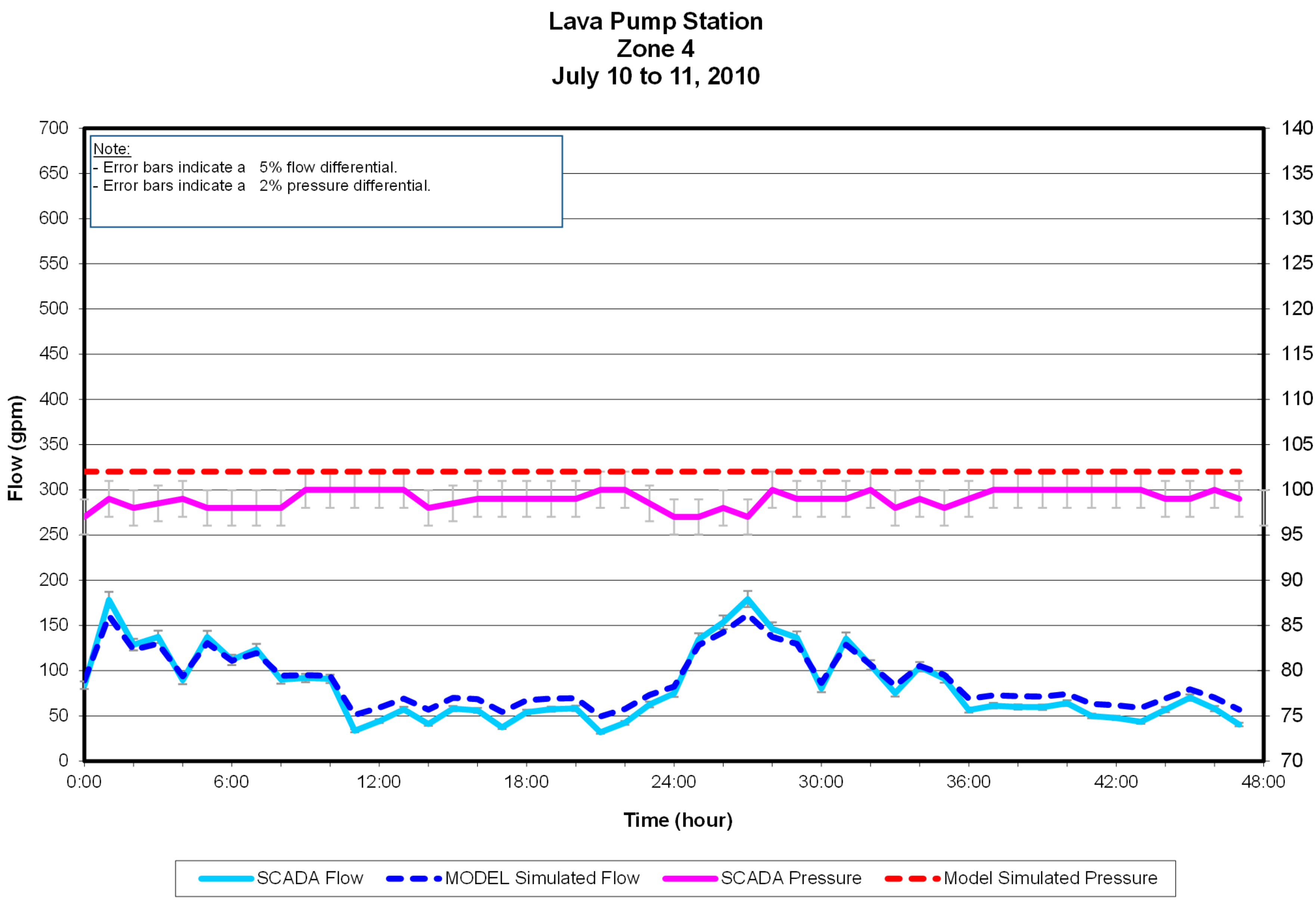
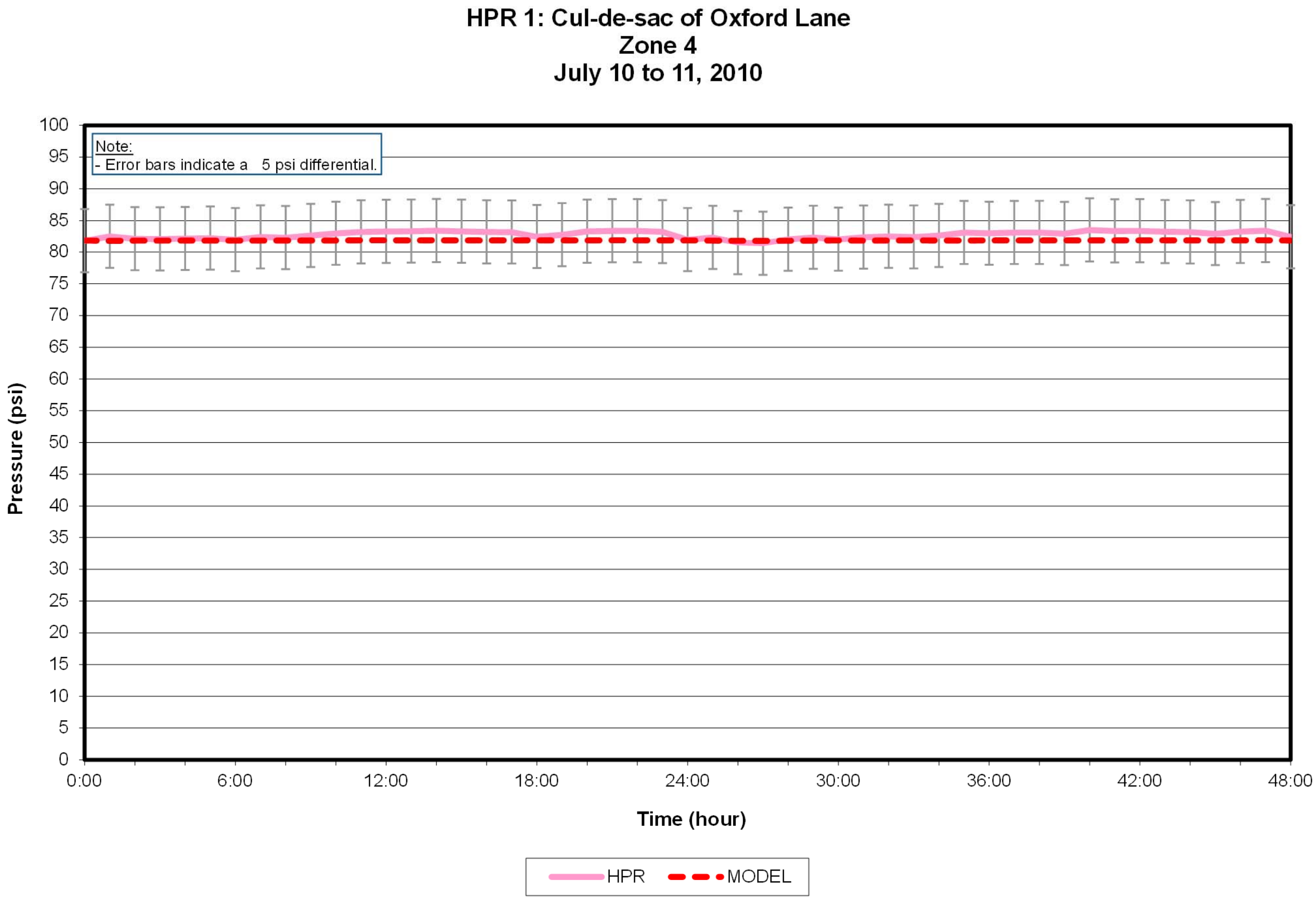
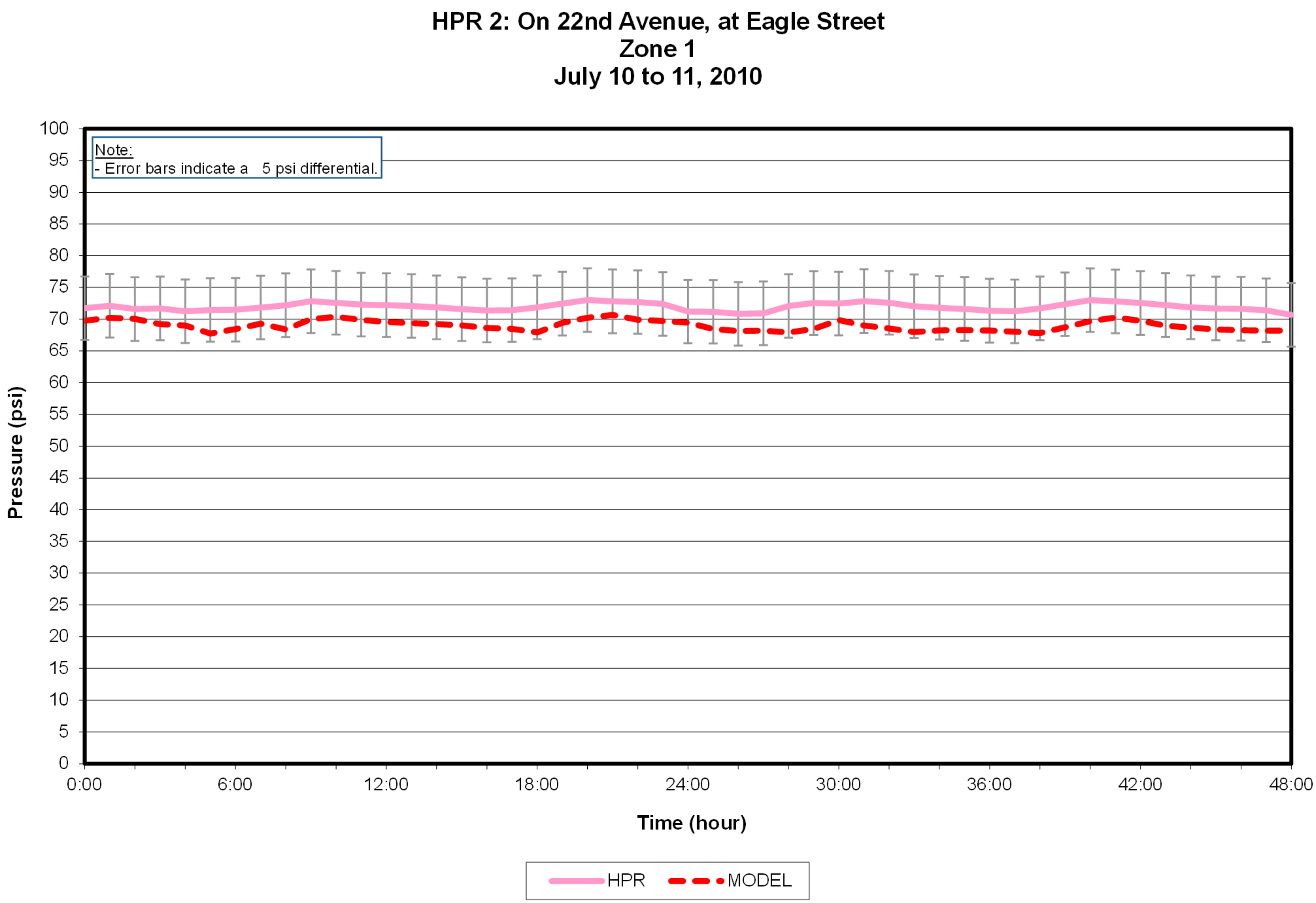
Last Revised: 11-17-10

City of Milwaukie
Water Master Plan

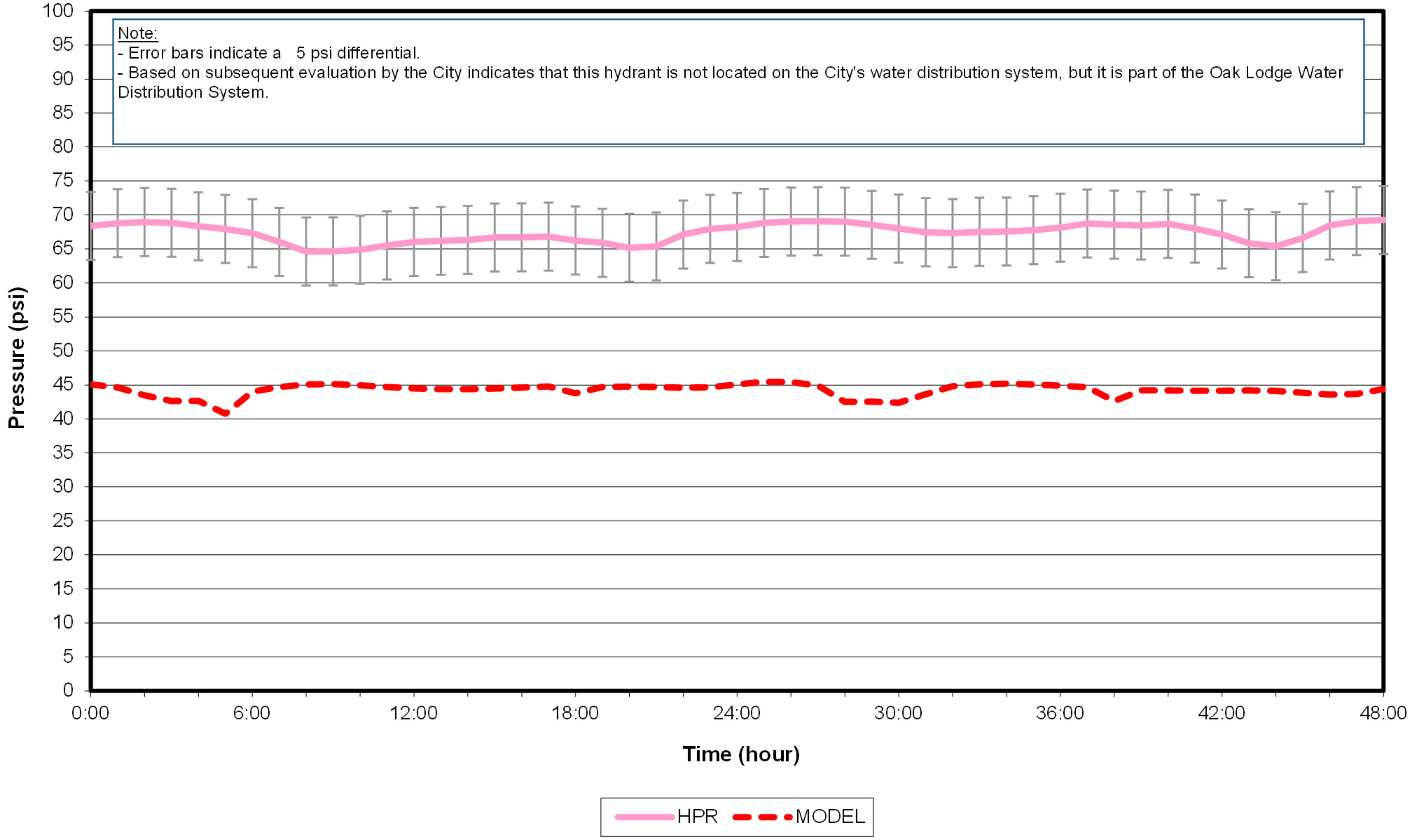


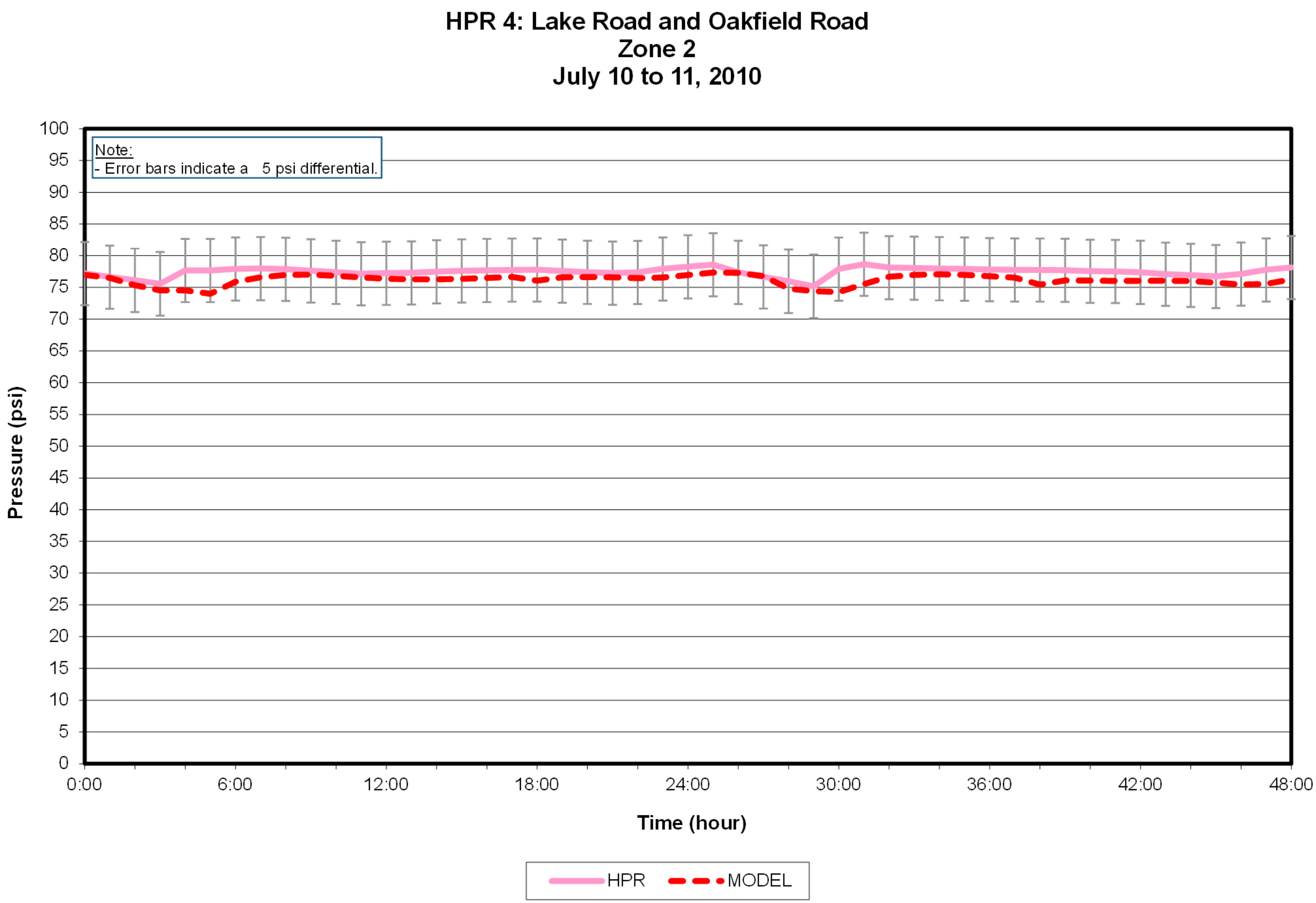
Lava Pump Station
Zone 4
July 10 to 11, 2010
Note:
- Error bars indicate a 5% flow differential.
- Error bars indicate a 2% pressure differential.
Flow (gpm)
700
650
600
550
500
450
400
350
300
250
200
150
100
50
0
140
135
130
125
120
115
110
105
100
95
90
85
80
75
70
0:00
6:00
12:00
18:00
24:00
30:00
36:00
42:00
48:00
Time (hour)
SCADA Flow
MODEL Simulated Flow
SCADA Pressure
Model Simulated Pressure


HPR 1: Cul-de-sac of Oxford Lane
Zone 4
July 10 to 11, 2010
Note:
- Error bars indicate a 5 psi differential.
Pressure (psi)
100
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
0:00
6:00
12:00
18:00
24:00
30:00
36:00
42:00
48:00
Time (hour)
HPR
MODEL


HPR 2: On 22nd Avenue, at Eagle Street
Zone 1
July 10 to 11, 2010
Note:
- Error bars indicate a 5 psi differential.
Pressure (psi)
100
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
0:00
6:00
12:00
18:00
24:00
30:00
36:00
42:00
48:00
Time (hour)
HPR
MODEL

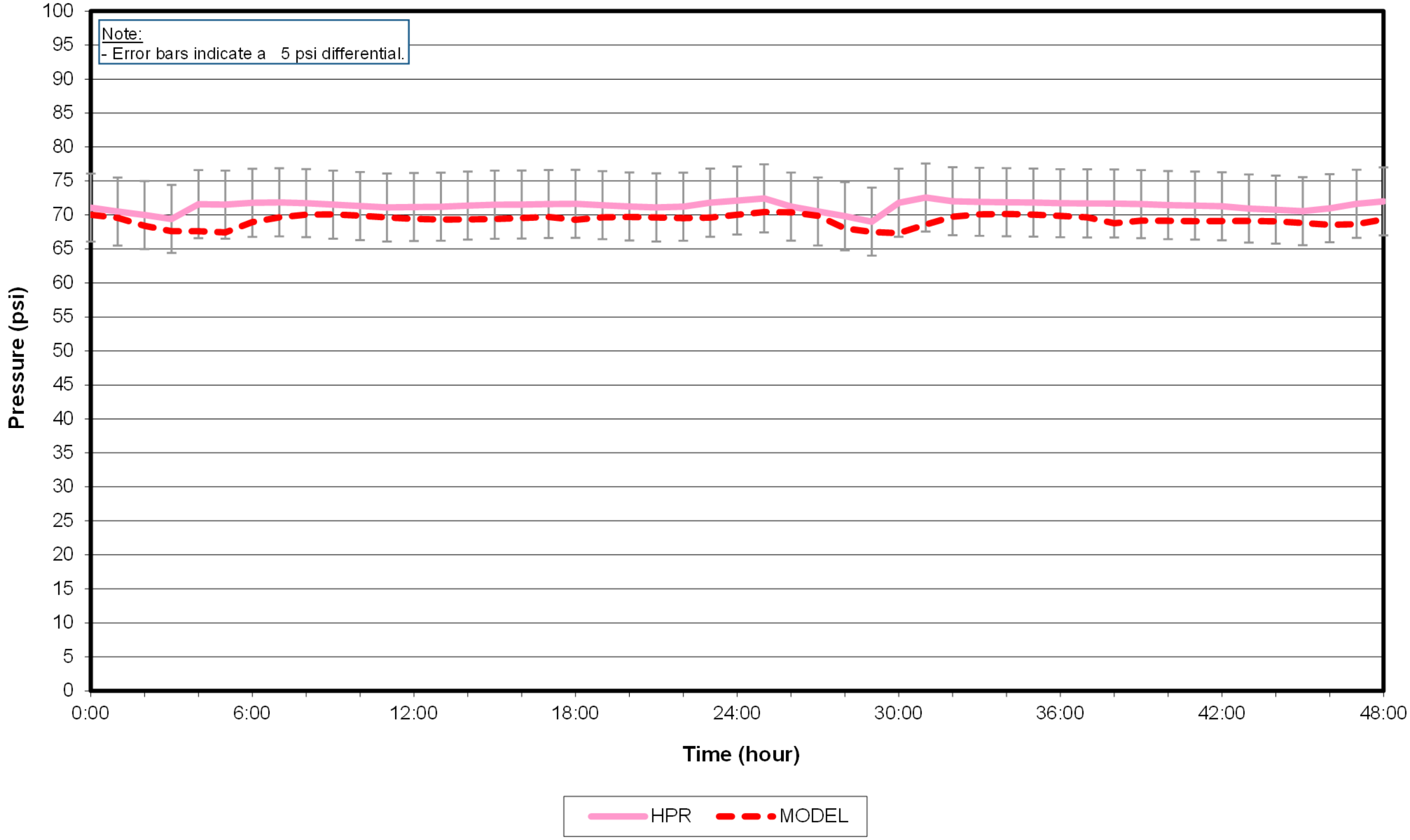
HPR 3: Dove Street and 24th Avenue
Zone 2
July 10 to 11, 2010



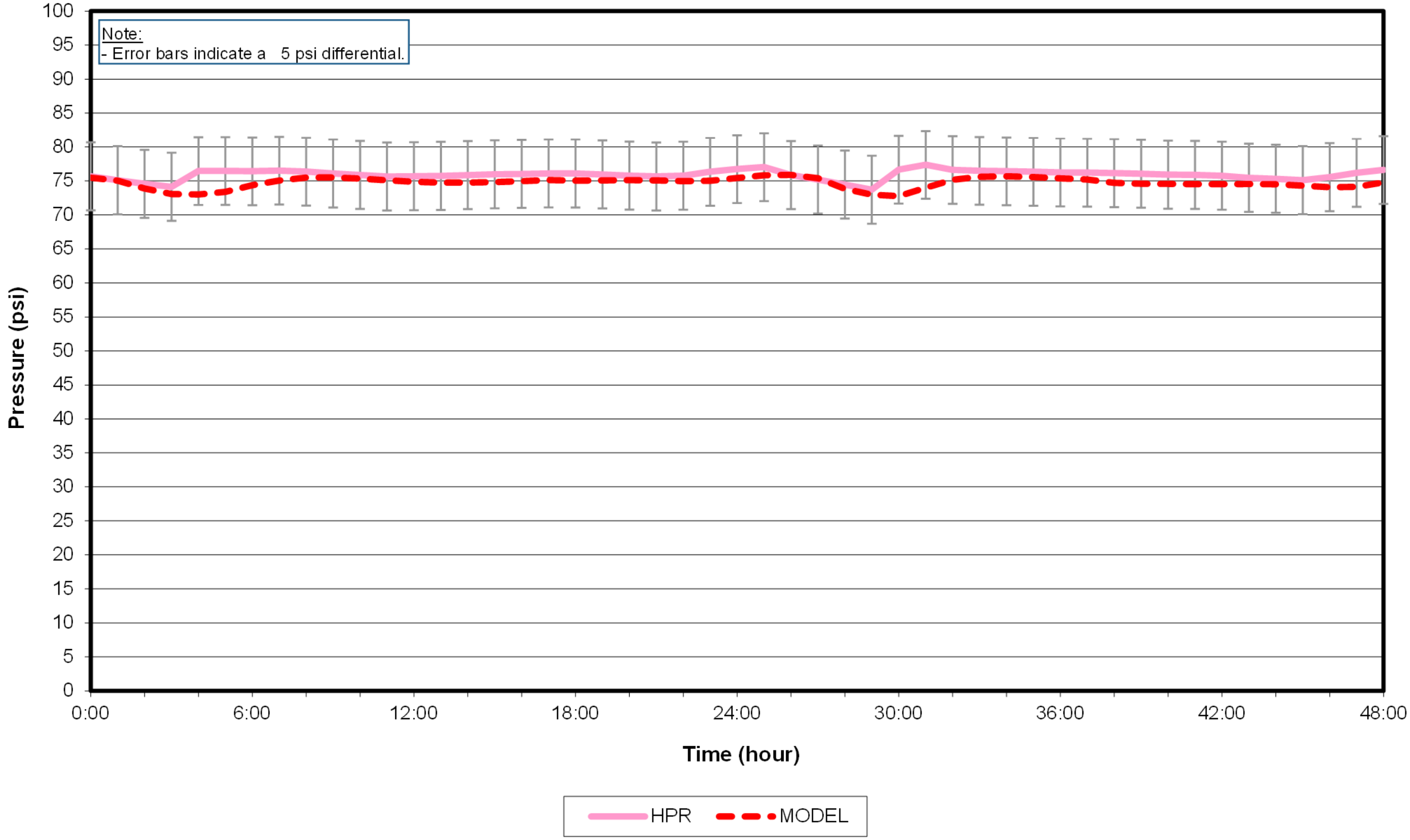


HPR 4: Lake Road and Oakfield Road
Zone 2
July 10 to 11, 2010
Note:
- Error bars indicate a 5 psi differential.
Pressure (psi)
100
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
0:00
6:00
12:00
18:00
24:00
30:00
36:00
42:00
48:00
Time (hour)
HPR
MODEL

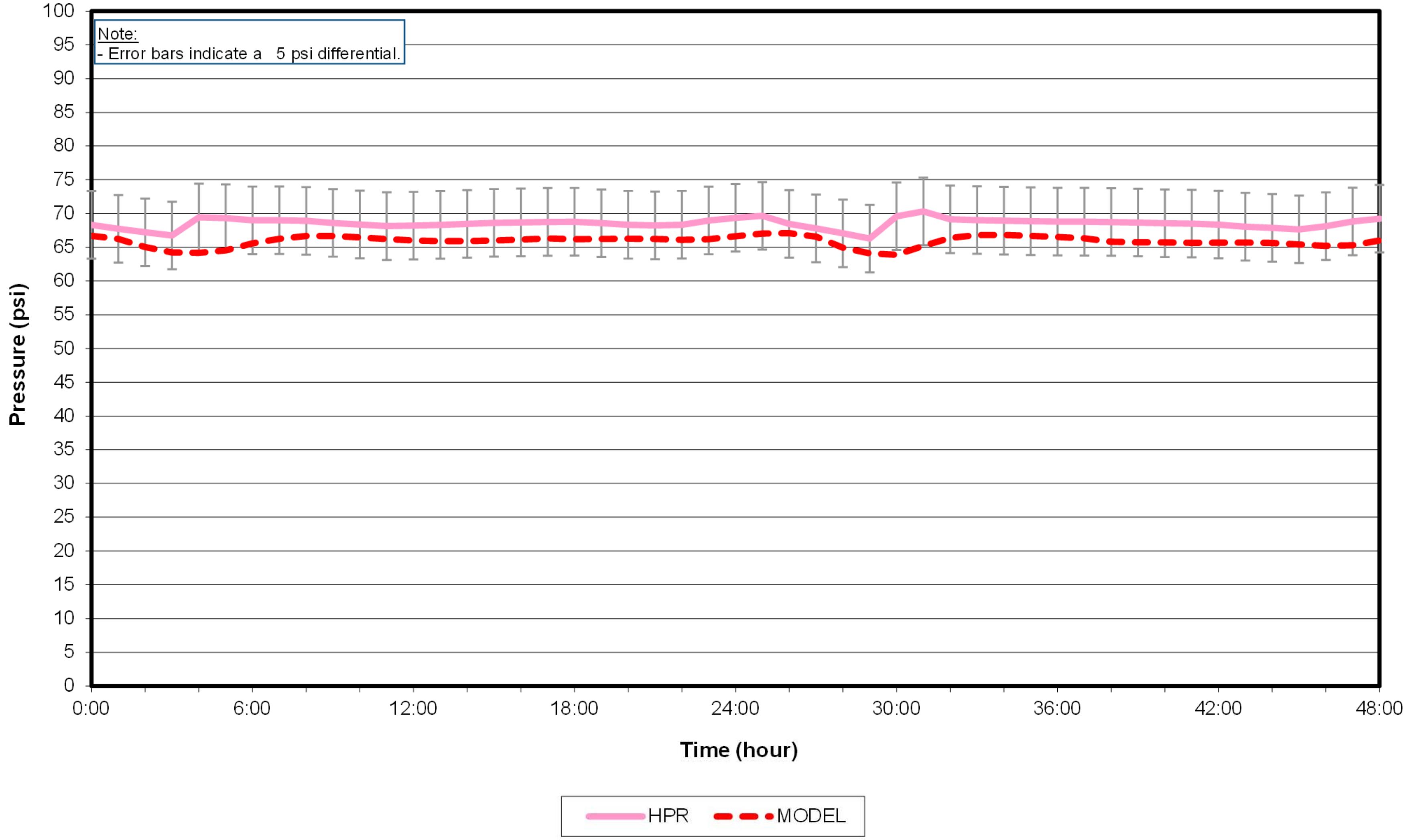
HPR 5: Lake Road, Between 41st Court and 43rd Avenue
Zone 2
July 10 to 11, 2010

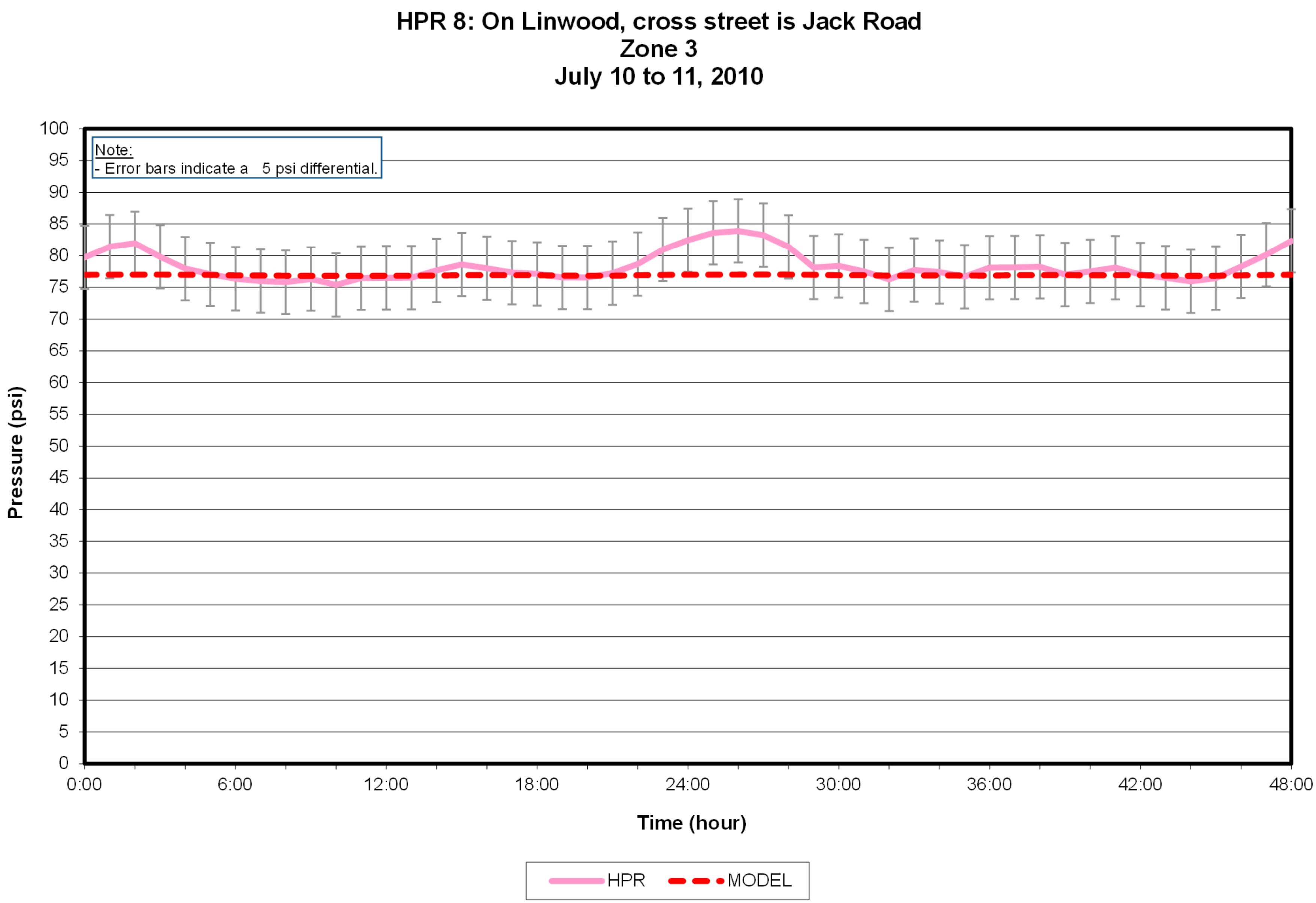


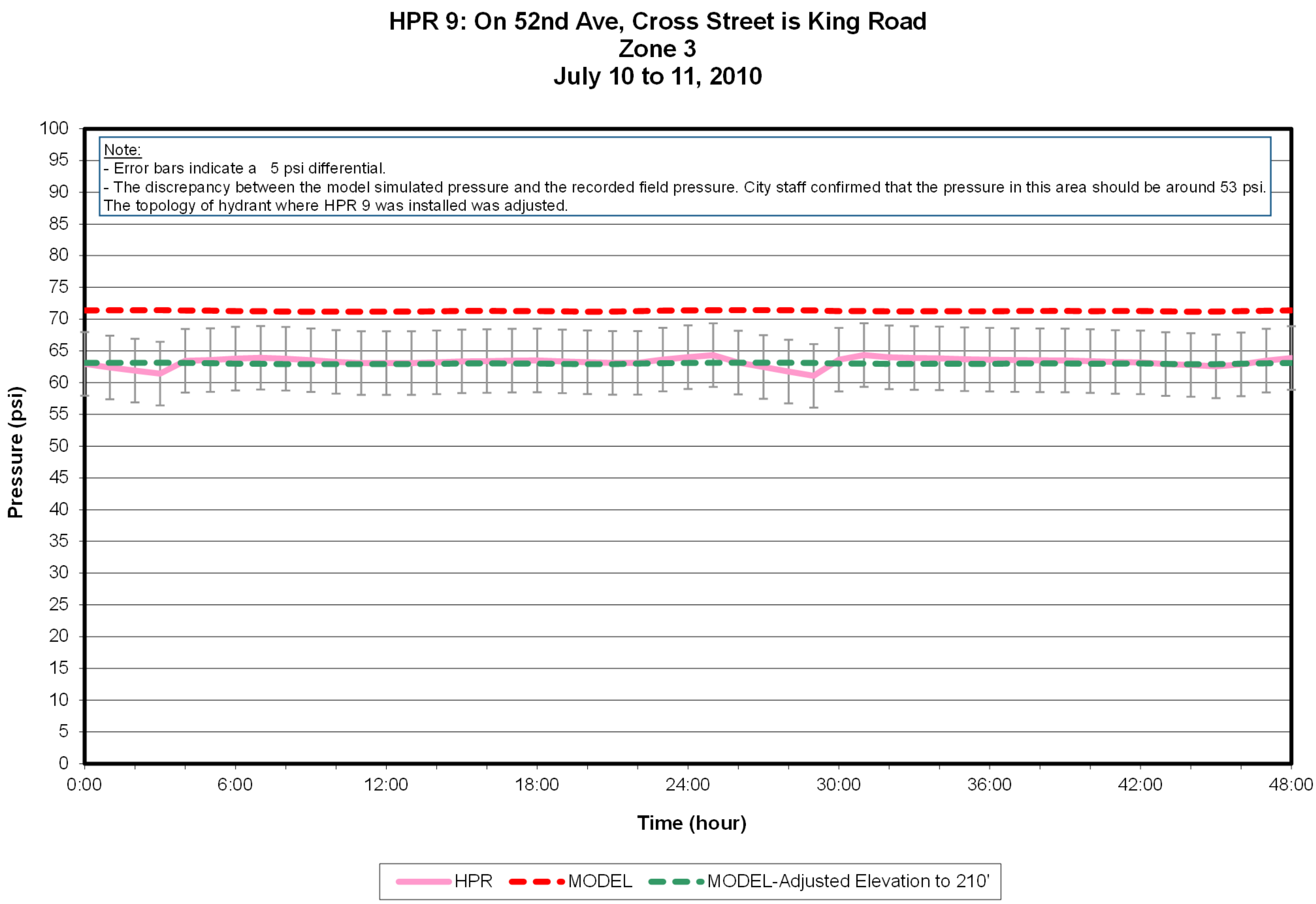
HPR 6: On Wood Avenue, at Railroad Avenue
Zone 2
July 10 to 11, 2010



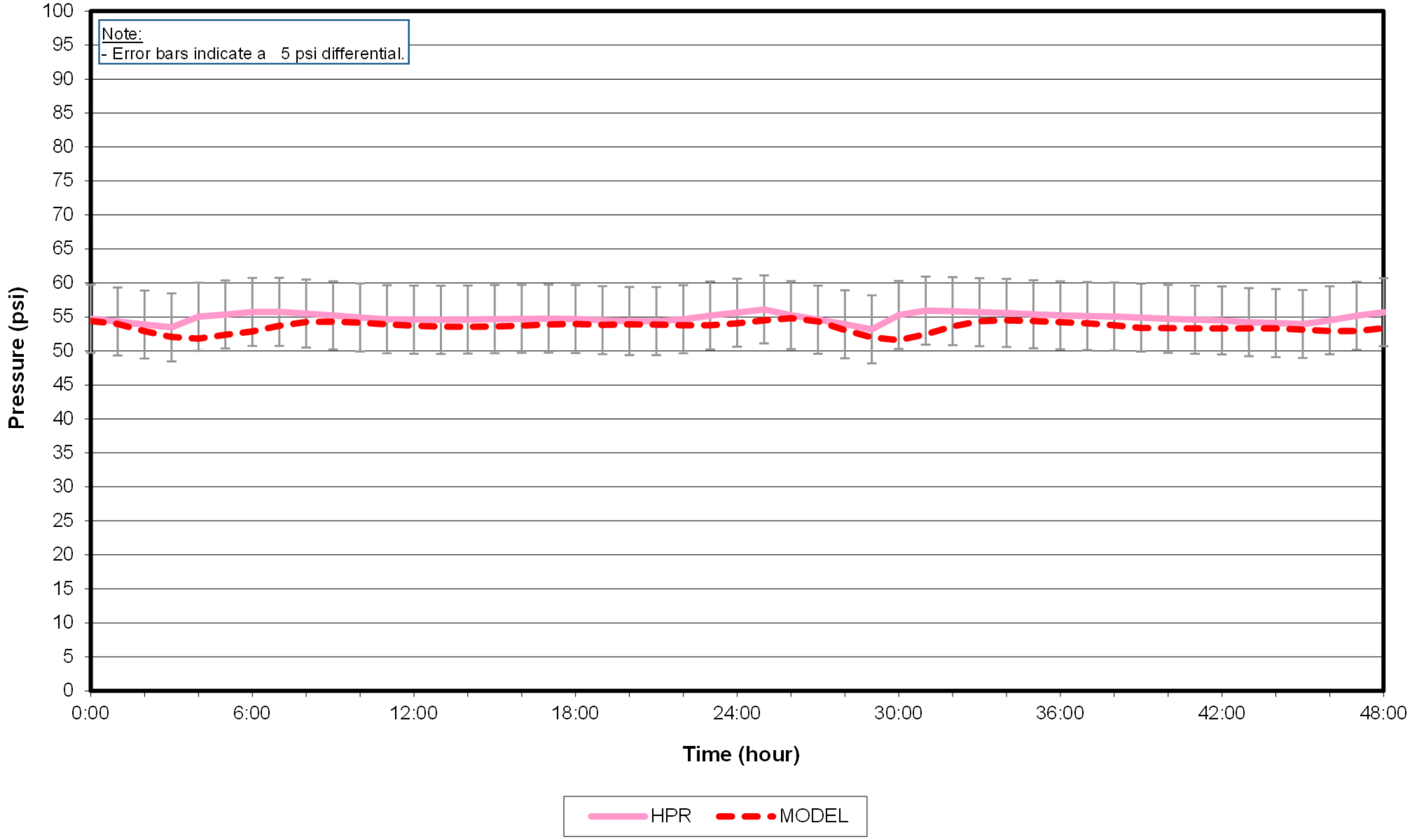
HPR 7: Madrona Drive, Between 70th and 71st Avenue
Zone 2
July 10 to 11, 2010



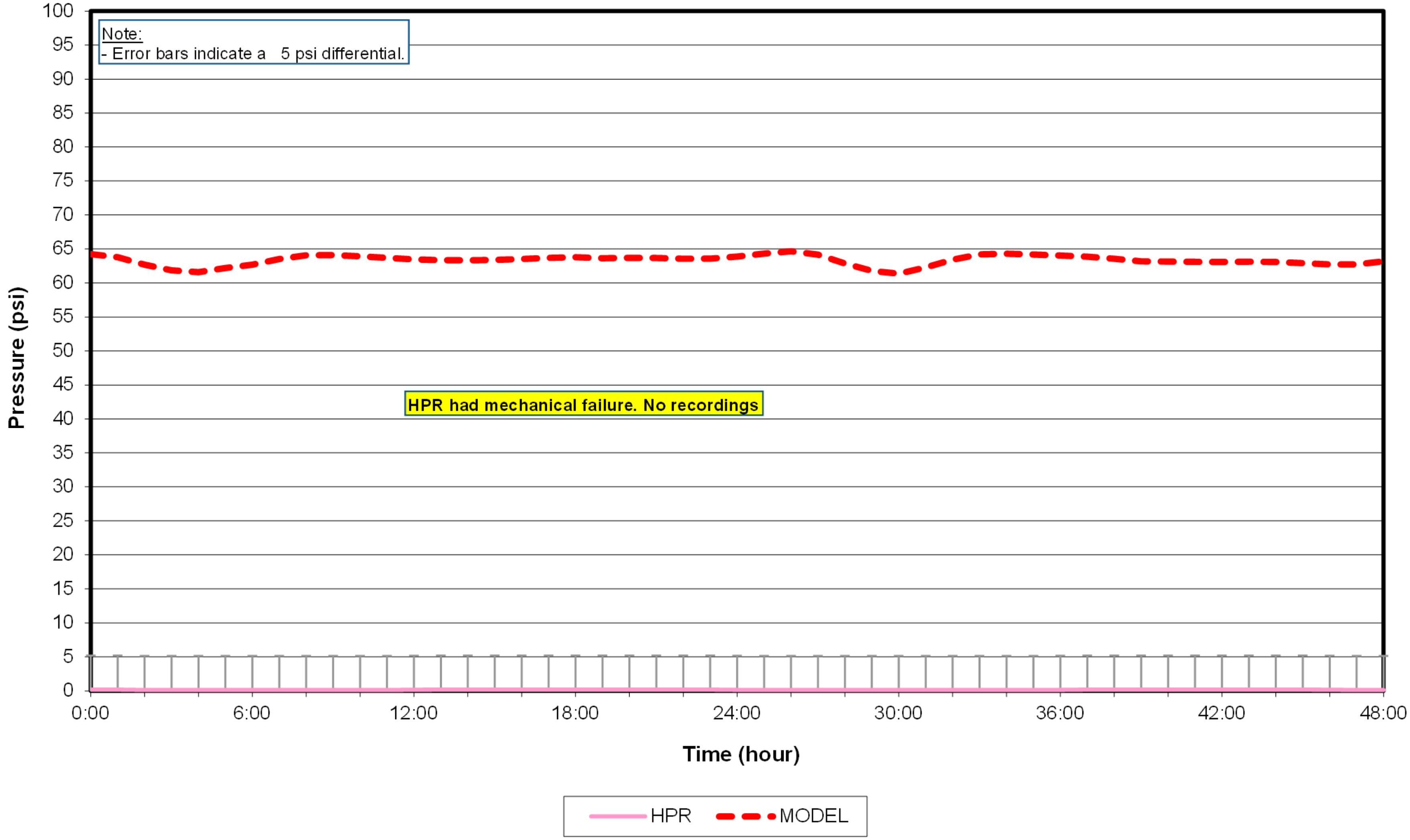

HPR 8: On Linwood, cross street is Jack Road
Zone 3
July 10 to 11, 2010
Note:
- Error bars indicate a 5 psi differential.
Pressure (psi)
100
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
0:00
6:00
12:00
18:00
24:00
30:00
36:00
42:00
48:00
Time (hour)
HPR
MODEL


HPR 9: On 52nd Ave, Cross Street is King Road
Zone 3
July 10 to 11, 2010
Note:
- Error bars indicate a 5 psi differential.
- The discrepancy between the model simulated pressure and the recorded field pressure. City staff confirmed that the pressure in this area should be around 53 psi. The topology of hydrant where HPR 9 was installed was adjusted.
Pressure (psi)
100
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
0:00
6:00
12:00
18:00
24:00
30:00
36:00
42:00
48:00
Time (hour)
HPR
MODEL
MODEL-Adjusted Elevation to 210'

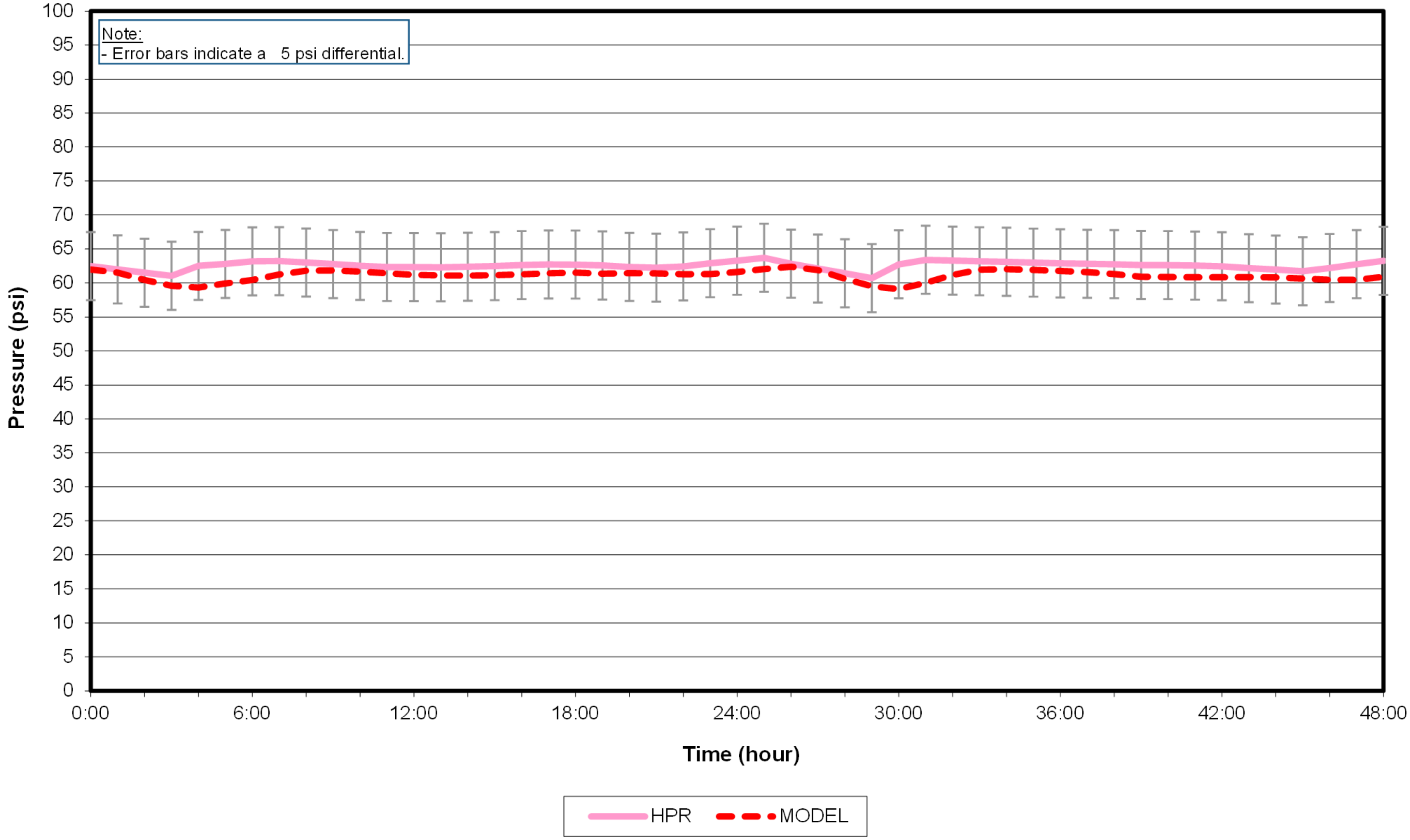
HPR 10: On Drefshill St, Cross Street is Stanley
Zone 2
July 10 to 11, 2010



HPR 11: Wichita Avenue, South of Johnson Creek Boulevard
Zone 2
July 10 to 11, 2010



HPR 12: 42nd Avenue, South of Johnson Creek Boulevard
Zone 2
July 10 to 11, 2010



WEST YOST ASSOCIATES
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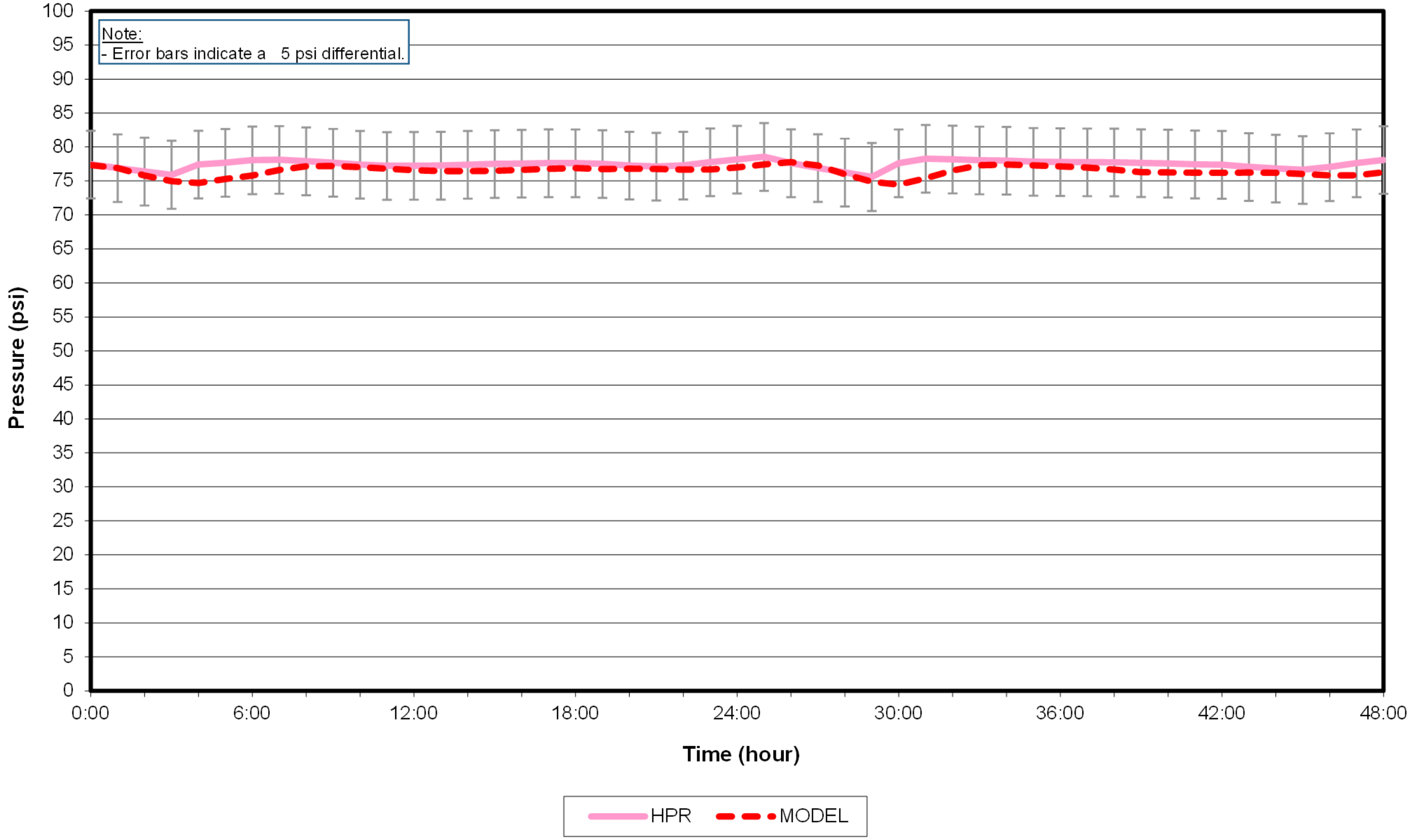
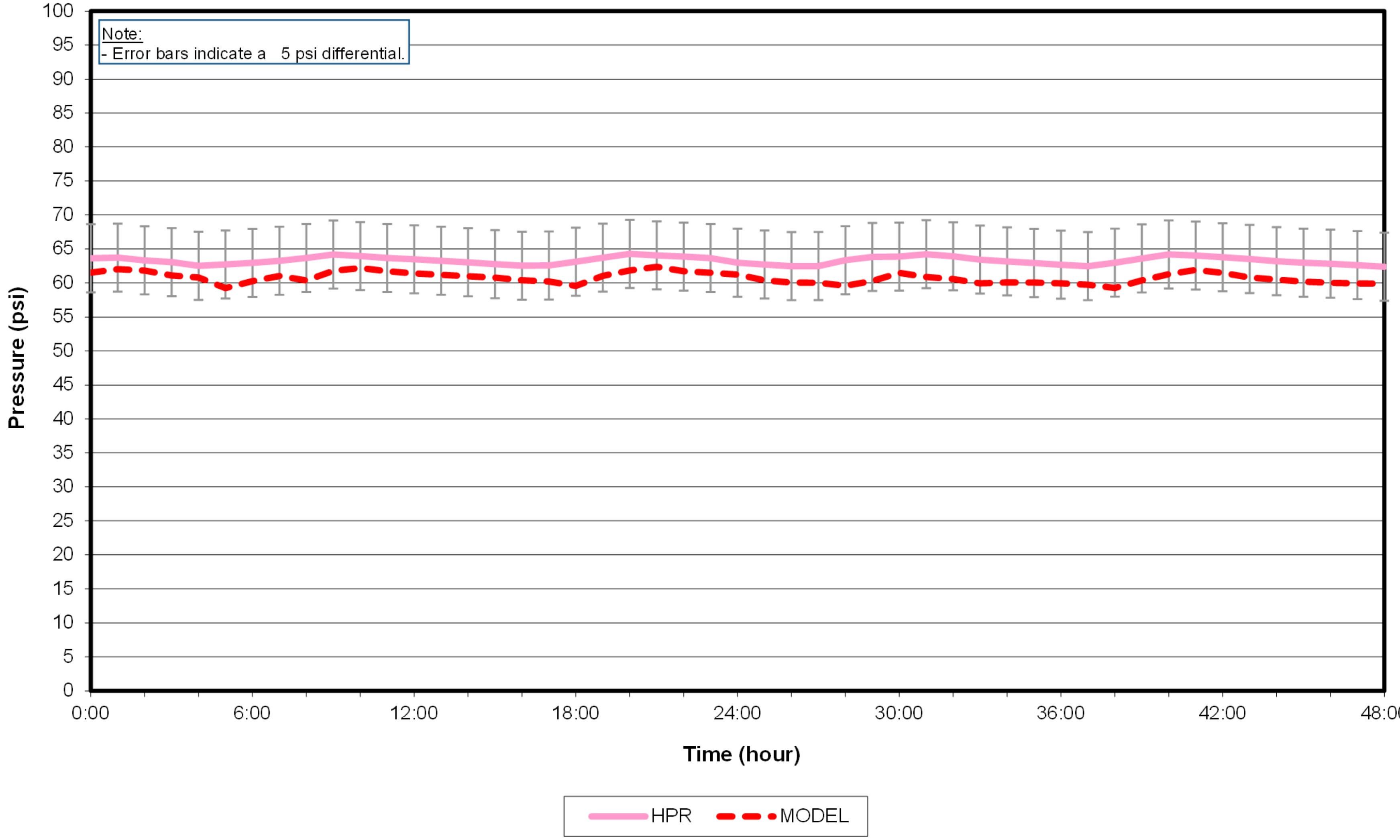
Last Revised: 11-17-10

City of Milwaukie
Water Master Plan

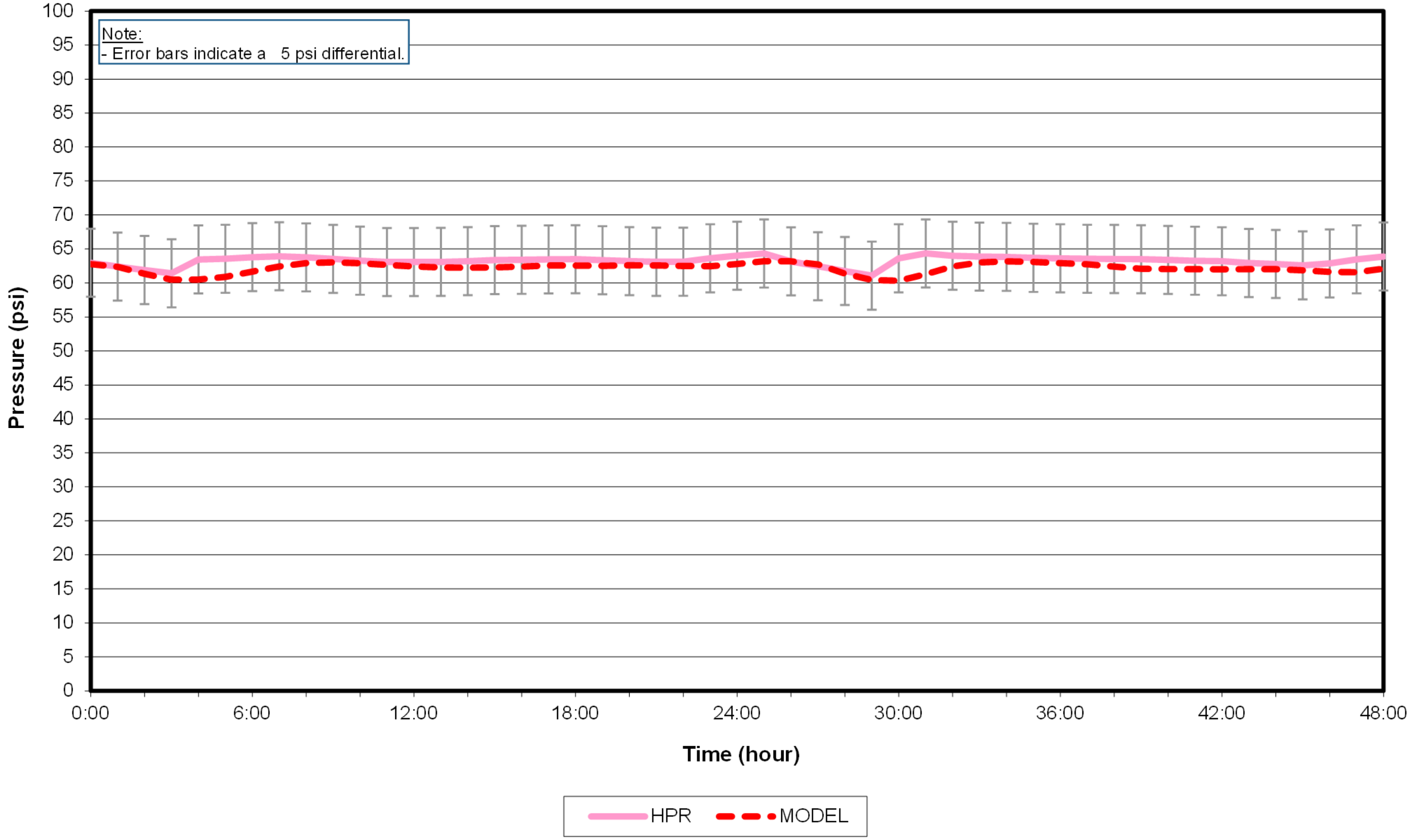
HPR 13: 28th Avenue and Van Water Street
Zone 2
July 10 to 11, 2010



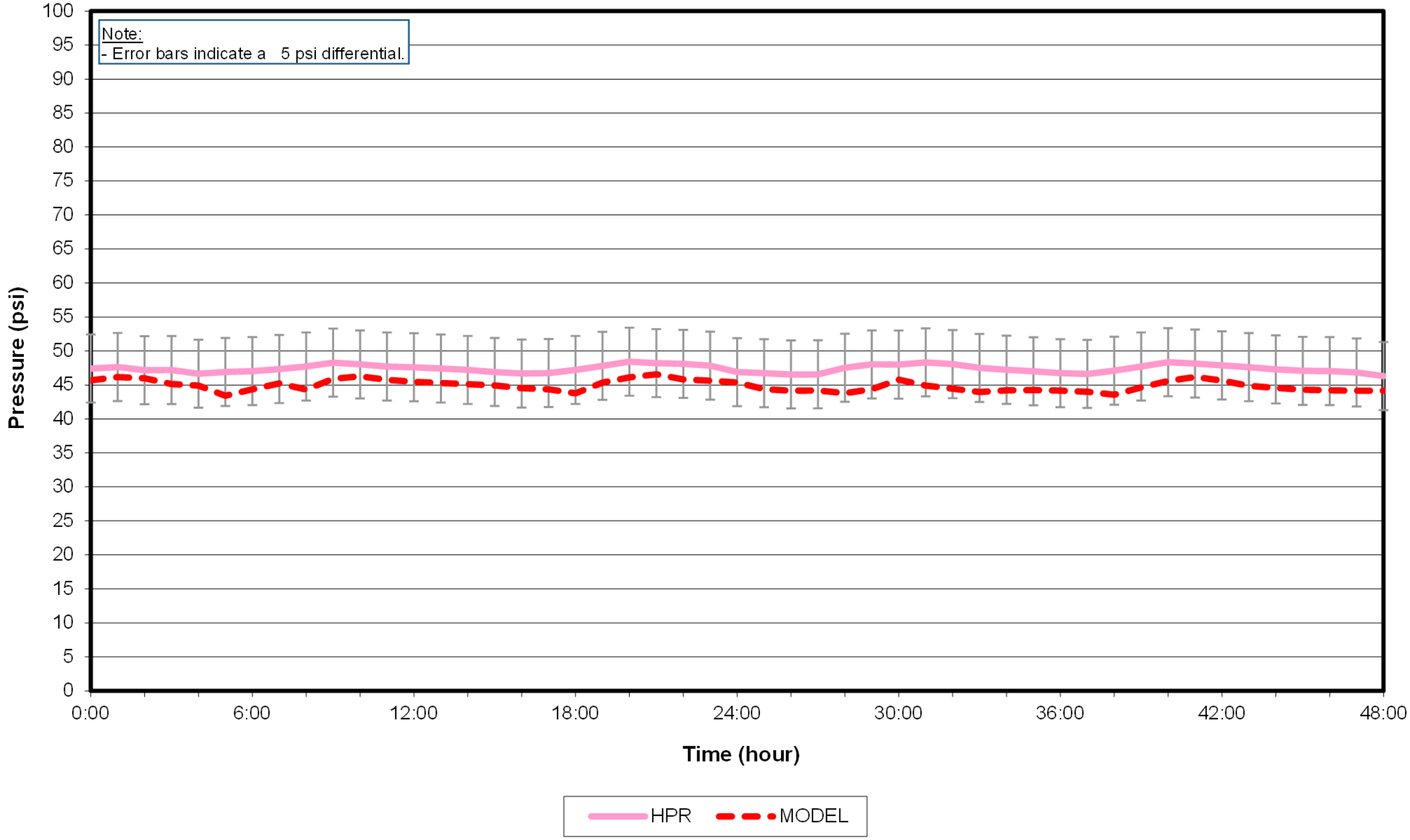
HPR 14: Intersection of Mailwell Drive and SPT Corridor
Zone 1
July 10 to 11, 2010



HPR 15: On Jackson Street, at 37th Avenue
Zone 2
July 10 to 11, 2010



HPR 16: 29th Avenue and Monroe Street
Zone 1
July 10 to 11, 2010



WEST YOST ASSOCIATES
o\c\382\03-10-01\e\t4\verif\HPR Verification_time Control Setting_matchingElevated Tank.xls
Last Revised: 11-17-10

City of Milwaukie
Water Master Plan

APPENDIX C

Cost Estimating Assumptions

APPENDIX C. COST ESTIMATING ASSUMPTIONS

This appendix provides the assumptions used by West Yost to estimate the construction costs for the planning and design of recommended water system facilities for the City of Milwaukie. The costs were developed based on data supplied by manufacturers, published industry standard cost data and curves, construction costs for similar facilities built by other public agencies, and construction costs previously estimated by West Yost for similar facilities with similar construction cost indexes.

Additionally, these costs are for construction only and do not include estimating uncertainties or unexpected construction costs (*e.g.*, variations in final quantities) or cost estimates for land acquisition, engineering, legal costs, environmental review, inspections and/or contract administration. These additional cost items are referred to as construction contingency costs and project cost allowances, and are further described in the last section of this appendix.

All construction costs have been adjusted to reflect January 2011 costs at an Engineering News Record (ENR) Construction Cost Index (CCI) of 8938 (20 Cities Average). These costs are to be used for conceptual cost estimates only, and should be updated regularly. Construction costs presented in this appendix are not intended to represent the lowest prices in the industry for each type of construction; rather they are representative of average or typical construction costs. The planning level cost estimates have been prepared for guidance in evaluating various options, and are intended for budgetary purposes only, within the context of this master planning effort.

CONSTRUCTION COSTS

Pipelines

Unit construction costs for potable water pipelines 6 through 36 inches in diameter are provided in Table 1. These costs are to be used for typical pipeline construction in developed areas and for construction across open fields or areas that are not yet developed (undeveloped). These costs generally include pipeline materials, trenching, placing and jointing pipe, valves, fittings, hydrants, service connections, placing imported pipe bedding, native backfill material, and asphalt pavement replacement, if required. The costs presented in Table 1 do not include the cost of boring and jacking pipe. The costs shown in Table 2 should be added where required for this purpose.

Table 1. Unit Construction Costs for Pipelines[a]

Pipe Diameter, inches	Unit Construction Cost, $/linear foot	
	Developed Areas	Undeveloped Areas
6	113	104
8	147	125
10	170	147
12	204	170
14	232	193
16	261	215
18	289	244
20	312	266
24	363	300
30	442	374
36	516	431

[a] Based on the January 2011 ENR index of 8938.

Table 2. Unit Construction Costs for Jack & Boring[a]

Size	Unit Construction Cost, $/linear foot[b]
8-inch pipe (16-inch casing)	408
12-inch pipe (21-inch casing)	465
16-inch pipe (24-inch casing)	538
20-inch pipe (30-inch casing)	663
54-inch pipe (66-inch casing)	1,331
Tunnel	2,776

[a] Based on the January 2011 ENR index of 8938.
[b] Conductor pipe not included in cost.

Treated Water Storage Reservoirs

Table 3 lists the estimated construction costs for water storage reservoirs between the size ranges of 0.1 to 6.0 MG. These costs generally include the storage tank, site piping, earthwork, paving, instrumentation, and all related sitework. As previously stated, these costs are representative of construction conducted under normal excavation and foundation conditions, and would be significantly higher for special or difficult foundation requirements.

Table 3. Construction Costs for Treated Water Storage Reservoirs[a]		
	Estimated Construction Cost, million dollars	
Capacity, MG	Partially Buried Pre-Stressed Concrete	Welded Steel
0.1	1.7	1.0
0.5	2.0	1.4
1.0	2.4	1.7
2.0	3.1	2.2
3.0	3.9	2.8
4.0	4.6	3.5
5.0	5.3	4.1
6.0	6.2	4.7

(a) Based on the January 2011 ENR index of 8938.

Treated Water Booster Pump Stations

Distribution pumping station costs vary considerably, depending on such factors as architectural design, pumping head, and station capacity. Estimated average construction costs for distribution pumping stations, as shown in Table 4, are based on enclosed stations with architectural and landscaping treatment suitable for residential areas. Booster pump station cost estimates include a chemical feed system (hypochlorite or fluoride), backup/standby generator plus SCADA, and are based on the typical Cal Water configuration, which includes 1 to 3 pumps at approximately 1 to 2 mgd.

Table 4. Construction Costs for Booster Pump Stations[a]	
Firm Capacity[b], mgd	Estimated Construction Cost, million dollars
0.5	1.0
1	1.0
2	1.2
3	1.4
5	1.6
10	2.2

(a) Based on the January 2011 ENR index of 8938.
(b) The pumping capacity with the largest pump out of service or on standby.

Groundwater Production Wells

Well construction consists of pilot hole drilling, water quality/soil sampling, pilot hole reaming, well construction, well development and providing the necessary housing, pump, motor, automatic control equipment, discharge piping, SCADA, and disinfection equipment. Costs are estimated to be approximately $1,257,000 per well. These costs are representative of construction conducted under normal drilling conditions, and would be significantly higher for special or difficult locations.

CONTINGENCIES AND OTHER PROJECT COSTS

Contingency costs must be reviewed on a case-by-case basis because they will vary considerably with each project. However, to assist the City with budgeting for these future construction projects, contingency costs have been added to the planning budget as percentages of the estimated construction cost using these two categories: Construction Contingency Costs and Other Project Cost Allowances.

Construction Contingency Costs

The construction costs presented above are representative of the construction of water system facilities under normal construction conditions and schedules; consequently, it is appropriate to allow for estimating and construction uncertainties unavoidably associated with the conceptual planning of projects. Factors such as unexpected construction conditions, the need for unforeseen mechanical items, and variations in final quantities are only a few of the items that can increase project costs for which it is wise to make allowances in these preliminary cost estimates. An allowance of 20 percent of the base construction cost will be included to cover such project related construction contingencies.

Other Project Cost Allowances

Other project cost allowances are divided into three subcategories, totaling 28 percent:

- Design services associated with new facilities include preliminary investigations and reports, right-of-way acquisition, foundation explorations, preparation of drawings and specifications for construction, surveying and staking, sampling of testing material, and start-up services. The cost of these items may vary, but for the purpose of this study, it is assumed that engineering design costs will equal 10 percent of the construction costs after construction contingencies have been applied.
- Construction management covers items such as contract management and inspection during construction. The cost of these items may vary, but for the purpose of this study, it is assumed that construction management costs will equal 10 percent of the construction costs after construction contingencies have been applied.
- Administration costs cover items such as legal fees, environmental/CEQA compliance requirements, financing expenses, and interest during construction. The cost of these items may vary, but for the purpose of this study, it is assumed that program implementation costs will equal 8 percent of the construction costs after construction contingencies have been applied.

An example application of these allowances to a project with an assumed base construction cost of $1.0 million is shown in Table 5. As shown, the total cost of all project construction contingencies (construction, design, construction management, and administration costs) is approximately 54 percent of the base construction cost for each project.

Table 5. Example Application of Construction Contingency Costs and Other Project Cost Allowances

Cost Component	Percent	Cost	Total Cost
Estimated Base Construction Cost before Contingencies		$1,000,000[a]	
Construction Contingency Costs	20%	200,000	
Estimated Construction Cost with Contingencies			*$1,200,000*
Other Project Cost Allowances:			
Design	10%	$120,000	
Construction Management	10%	120,000	
Administration	8%	96,000	
Total Project Cost Allowances			$336,000
Estimated Total Project Cost			**$1,536,000**

[a] Assumed cost of example project.