DRAFT REPORT | JUNE 2023

Water Master Plan

PREPARED FOR

City of Sweet Home

Sweet Home
Oregon at its best!

PREPARED BY

WEST YOST
Water. Engineered.

Water Master Plan

Prepared for

City of Sweet Home

Project No. 936-60-20-21

Project Manager: Preston Van Meter	Date
QA/QC Review: Polly Boissevain	Date

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LIST OF ACRONYMS AND ABBREVIATIONS

µg/L	Microgram per Liter
AAGR	Annual Average Growth Rate
ACE	ACE Engineering LLC
ADD	Average Day Demand
AF	Acre-Feet
ALA	American Lifelines Alliance
ATS	Automatic Transfer Switch
AWWA	American Water Works Association
BP	Backwash Pump
C/R	Capacity or Reliability Improvements
cfs	Cubic Feet Squared
CI	Cast Iron
CIP	Capital Improvement Plan
City	City of Sweet Home
CMU	Concrete Masonry Unit
County	Linn County
CSZ	Cascadia Subduction Zone
DBPR	Disinfection Byproducts Rule
DEM	Digital Elevation Model

DI	Ductile Iron
DIP	Ductile Iron Pipe
DOGAMI	Department of Geology and Mineral Industries
DWS	Drinking Water Services
ELA	Engineering, Legal, and Administrative Services
ELISA	Enzyme-Linked Immunosorbent Assay
ENR CCI	Engineering News Record Construction Cost Index
EPA	Environmental Protection Agency
FFI	Fire Flow Improvements
fps	Feet Per Second
FW	Fresh Water
GALV	Galvanized Steel
GIS	Geographic Information System
gpcd	Gallons Per Capita Per Day
gpm	Gallons Per Minute
HABs	Harmful Algal Blooms
HDPE	High Density Polyethylene
hp	Horsepower
IDSE	Initial Distribution System Evaluation
IOC	Inorganic Carbon
LCAA	Locational Running Annual Average
Lidar	Light Detection and Ranging
M	Million
MCE_R	Maximum Considered Earthquake
MCL	Maximum Contaminant Level
MDD	Maximum Day Demand
MG	Million Gallons
MG/yr	Million Gallons Per Year
mgd	Million Gallons Per Day
msl	Mean Sea Level
NRW	Non-Revenue Water
O&M	Operation and Maintenance
OARs	Oregon Administrative Rules
OFC	Oregon Fire Code
OHA	Oregon Health Authority
ORP	Oregon Resilience Plan
ORWD	Oregon Water Resources Department
OSSPAC	Oregon Seismic Safety Policy Advisory Commission
PGD	Permanent Ground Deformations
PGV	Peak Ground Velocity
PHD	Peak Hour Demand
PRC	Population Research Center

PRVs	Pressure Reducing Valves
PS	Pump Station
psi	Pounds Per Square Inch
PSU	Portland State University
PVC	Polyvinyl Chloride
RLDWA	Reduction of Lead in Drinking Water Act
RR	Repair Rate
RTCR	Revised Total Coliform Rule
SCADA	Supervisory Control and Data Acquisition
SDC	System Development Charge
SDM Program	Small Diameter Water Main Replacement Program
SDWA	Safe Drinking Water Act
STL	Steel
TDH	Total Dynamic Head
TOC	Total Organic Carbon
TTHM	Trihalomethanes
UGB	Urban Growth Boundary
UPC	Uniform Plumbing Code
US	United States
USACE	US Army Corps of Engineers
Valley	Willamette Valley
VFD	Variable Frequency Drive
WMP	Water Master Plan
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant

Executive Summary

INTRODUCTION (CHAPTER 1)

This Water Master Plan (WMP) for the City of Sweet Home (City) formulates a comprehensive, current Capital Improvement Program (CIP) that can serve as a roadmap to meet the needs of the City's existing and future water customers. In 2016, the City completed a combined Water Management and Conservation Plan and WMP. Since the City's previous WMP was developed, the City has implemented many of the recommended CIP projects and has completed significant water system improvement projects throughout the system. Therefore, this WMP serves to evaluate the current water system under existing and future demand conditions, identify any existing system deficiencies, and recommend water system improvements.

The objectives of this WMP are to:

- Evaluate historical water meter data to develop current and estimated future water system average and peak demands;
- Identify design, operational, and performance criteria to guide the water system evaluations;
- Update the City's Geographic Information System (GIS)-based water system hydraulic model and re-allocate recent demands to the hydraulic model;
- Analyze the existing distribution system to evaluate the ability of the City's water system to meet current and future demands using the water system hydraulic model;
- Evaluate the existing water treatment plant (WTP) for hydraulic capacity and to identify operation and maintenance (O&M) needs;
- Prepare a seismic resiliency analysis to evaluate seismic hazards and their potential impact on the water system;
- Identify system deficiencies and recommend upgrades to meet operational and performance criteria; and,
- Develop a comprehensive CIP to address existing system deficiencies.

EXISTING SYSTEM DESCRIPTION (CHAPTER 2)

The City is located within Linn County (County), Oregon, about 75 miles south of Portland, 40 miles southeast of Salem, and 30 miles northeast of Eugene. The existing water service area is approximately 3.65 square miles and is generally contiguous with the City limits. The City's service area includes three pressure zones (Main, Strawberry, and LakePointe) and is served by approximately 54 miles of distribution pipelines, five storage tanks, and three booster pump stations.

The City's existing water supply portfolio includes surface water from the South Santiam River, which is impounded at the Foster Reservoir, and Ames Creek. The City has four existing water rights: two fully perfected and one partially perfected water rights permits to the South Santiam River and one perfected water rights permit to the Ames Creek. The City's primary water supply is surface water from the South Santiam River. At the time of this WMP, the City does not divert water from Ames Creek. The City diverts South Santiam River water from the Foster Reservoir and conveys the raw water to the City's WTP for treatment.

WATER DEMAND (CHAPTER 3)

The City's water service area is generally contiguous with the City limits. The City has a current population of 9,400, with population projected to grow to 12,800 by 2043, the 20-year horizon of this WMP. The City utilizes surface water from Foster Reservoir as the primary potable water sources and treats it at the City's WTP before distributing it to the water system. The City's historical water production has averaged 311 million gallons per year (MG/yr) for the period from 2016 through 2020, equivalent to an average daily production of 0.85 million gallons per day (mgd).

The City's average daily water use is expected to increase to 1.10 mgd by 2043 due to population growth and future development distributed throughout the City limits and the City's Urban Growth Boundary (UGB). Projected water demands were proportionally distributed among the buildable vacant parcels and future developments based on the parcel's and/or project's area.

DESIGN AND PERFORMANCE CRITERIA (CHAPTER 4)

Chapter 4 defines the recommended design and planning to be used for evaluating the performance of the City's water distribution system and planning for future growth. Recommended design and planning criteria include fire flow criteria, water supply and treatment capacity, allowable distribution system pressures, booster pump station capacity, water storage capacity, and pipeline sizing criteria. These criteria are used to identify system deficiencies and to size required improvements. The City is also responsible for ensuring that the applicable water quality standards and regulations established by the Oregon Health Authority (OHA) are met.

HYDRAULIC MODEL UPDATE (CHAPTER 5)

The City's distribution system hydraulic model was updated using the most current records provided by the City for pipelines and major facilities. Average day water demands for calendar year 2020 were allocated in the hydraulic model by pressure zone using the spatially-located meter account data. West Yost calibrated the hydraulic model using data gathered from a hydrant testing program conducted in January 2022. In updating the model, West Yost worked closely with the City's Public Works Department staff to assure accuracy of the model. Based on the results of the model calibration, it can be concluded that the hydraulic model provides a reasonable representation of the City's water distribution system and can be used as a tool for master planning purposes.

WATER SYSTEM ANALYSIS (CHAPTER 6)

Chapter 6 presents an analysis of the City's existing and future water system and its ability to meet recommended water service and performance standards under future demands for the 20-year master plan horizon. The analysis includes both system capacity and hydraulic performance evaluations based on the performance criteria presented in Chapter 4.

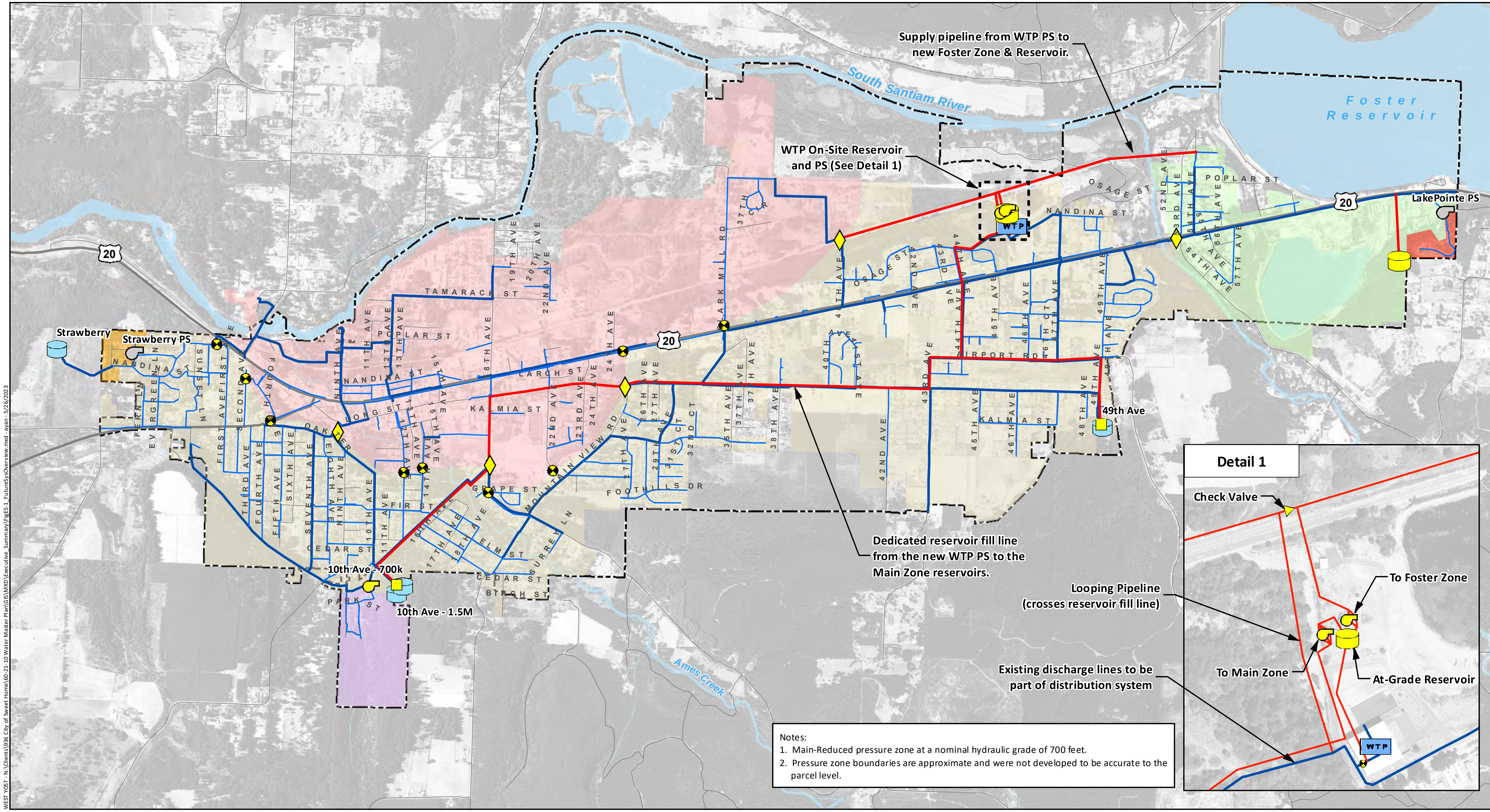
System Capacity Analysis

The system capacity analysis evaluates the City's existing and future water system facilities and their ability to meet the City's recommended performance and planning criteria under existing and projected water demand conditions. This analysis evaluates supply, pumping, and storage capacity needs to meet system requirements. The system capacity analysis found that City's system requires additional pumping capacity and storage capacity to meet existing and future demands.

Initial discussions of proposed water system improvements with the City indicated the need for major system configuration changes. This configuration is the basis for the future system capacity evaluation. The key proposed changes to the City's system are summarized in Table ES-1 and shown on Figure ES-1:

Table ES-1. Summary of Proposed Water System Improvements

Improvement	Description
Improvements in Main Pressure Zone	• Reconfigure the northwest portion of the Main Zone to supply the lower elevation areas of the pressure zone via pressure reducing valves (PRVs), creating the proposed Main-Reduced Zone to alleviate high pressures. • Install an at grade finished water reservoir at the WTP with a pump station to pump into the Main Zone. • Install a dedicated transmission pipeline direct from the new WTP pump station to the Main Zone reservoirs to improve zone operations. • Install altitude valves at the Main Zone reservoirs to improve tank operations.
Improvements East of Wiley Creek	• Install pumps at the new WTP pump station to a new supply pipeline parallel to the existing railroad north of the WTP, creating the proposed Foster Zone to alleviate low pressures and provide redundancy to the area. • Construct a new storage reservoir for the proposed Foster Zone, sited in the undeveloped hills immediately west of the LakePointe Zone.
Improvements South of 10th Avenue	• Construct a new pump station sited near southern terminus of 10th Avenue, which would supply a new closed pressure zone, the proposed 10th Avenue Zone.



Recommended Pressure Zones
- Strawberry
- LakePointe
- Main
- Main-Reduced (New)
- Foster (New)
- 10th Ave (New)

WTP Existing Water Treatment Plant
Existing Storage Tank
Potable Water Pump Station

Existing System Pipelines
- Diameter Less than 10-inches
- Diameter 10-inches and Greater

Recommended Pump Station
Recommended Storage Tank
Recommended Normally Closed Valve
Recommended Altitude Valve
Recommended Pressure Reducing Valve

Required New Pipeline for Recommended Operations
City Limit

N

0 900 1,800
Scale in Feet

WEST YOST

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Figure ES-1

Operational Overview of Recommended Future System

City of Sweet Home
Water Master Plan

System Performance Analysis

Hydraulic evaluations were performed using the City's updated hydraulic model to assess the performance of the water distribution system under future water demand conditions, first for the existing distribution system configuration, to identify deficiencies, and then with the future water system configuration, to identify any improvements needed in addition to reconfiguration improvements. The performance evaluation assesses the water system's ability to meet recommended performance standards under future peak hour demand conditions and future maximum day demand plus fire flow.

The existing system performance analysis found that the City's existing water system generally meets the performance criteria under normal operations, except for low pressures in the areas north and southwest of the 49th Avenue Reservoir, along Santiam Highway, and the area southwest of the 10th Avenue Reservoirs. A large portion of the City's system (i.e., areas with large fire flow requirements, hydrants on 2-inch diameter pipelines, long dead-end pipelines, etc.) cannot provide sufficient fire flow to satisfy the City's fire flow criteria.

Results of the future system performance analysis show that the City's future system generally resolves most of the issues described above, indicating that the major system configuration changes identified by the City in Table ES-1 are needed to address system deficiencies.

Summary of Recommended Improvements

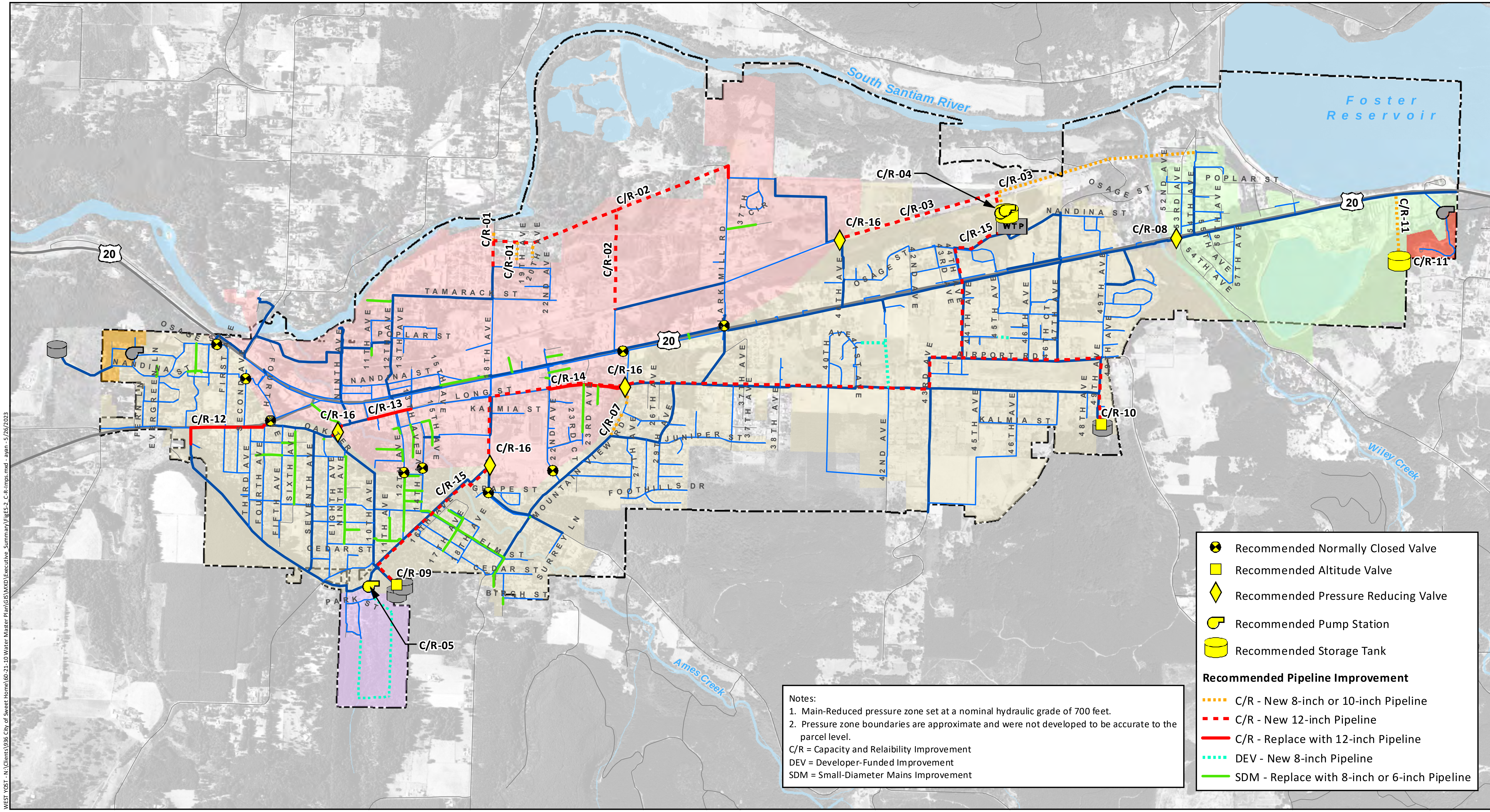
A summary of the recommended improvements proposed to eliminate the water system capacity and performance deficiencies identified in the preceding evaluations are categorized as Small Diameter Mains Improvements, Capacity or Reliability Improvements, and Fire Flow Improvements. Figures ES-2 and ES-3 illustrate the locations of the recommended Capacity and Reliability, Fire Flow and Small Diameter Mains improvement projects.

WATER TREATMENT PLANT EVALUATION AND UPGRADES (CHAPTER 7)

West Yost evaluated the City's existing WTP system capacity and performance and identified needs for meeting water service requirements and performance standards over the 20-year master planning horizon. The results of the system capacity evaluation indicate that the existing WTP has more than sufficient capacity to meet current and future demands over the 20-year master planning horizon. The firm capacity of the WTP is approximately 4.0 mgd compared with current and projected required maximum day production of 2.0 mgd and 2.6 mgd, respectively.

Additionally, West Yost conducted a condition assessment of the WTP with City staff to identify any potential deficiencies in the treatment process. The WTP improvements identified from the condition assessment are as follows:

- WTP Project #1: Filter Feed Manifold Piping Upgrades
- WTP Project #2: New Standby Generator and ATS
- WTP Project #3: Filter Sludge Removal System Replacement
- WTP Project #4: New Sludge Drying Bed



Recommended Pressure Zones

- Strawberry
- LakePointe
- Main
- Main-Reduced (New)
- Foster (New)
- 10th Ave (New)

WTP Existing Water Treatment Plant

City Limit

Existing Storage Tank

Potable Water Pump Station

Existing System Pipelines

- Diameter Less than 10-inches
- Diameter 10-inches and Greater

DRAFT

N

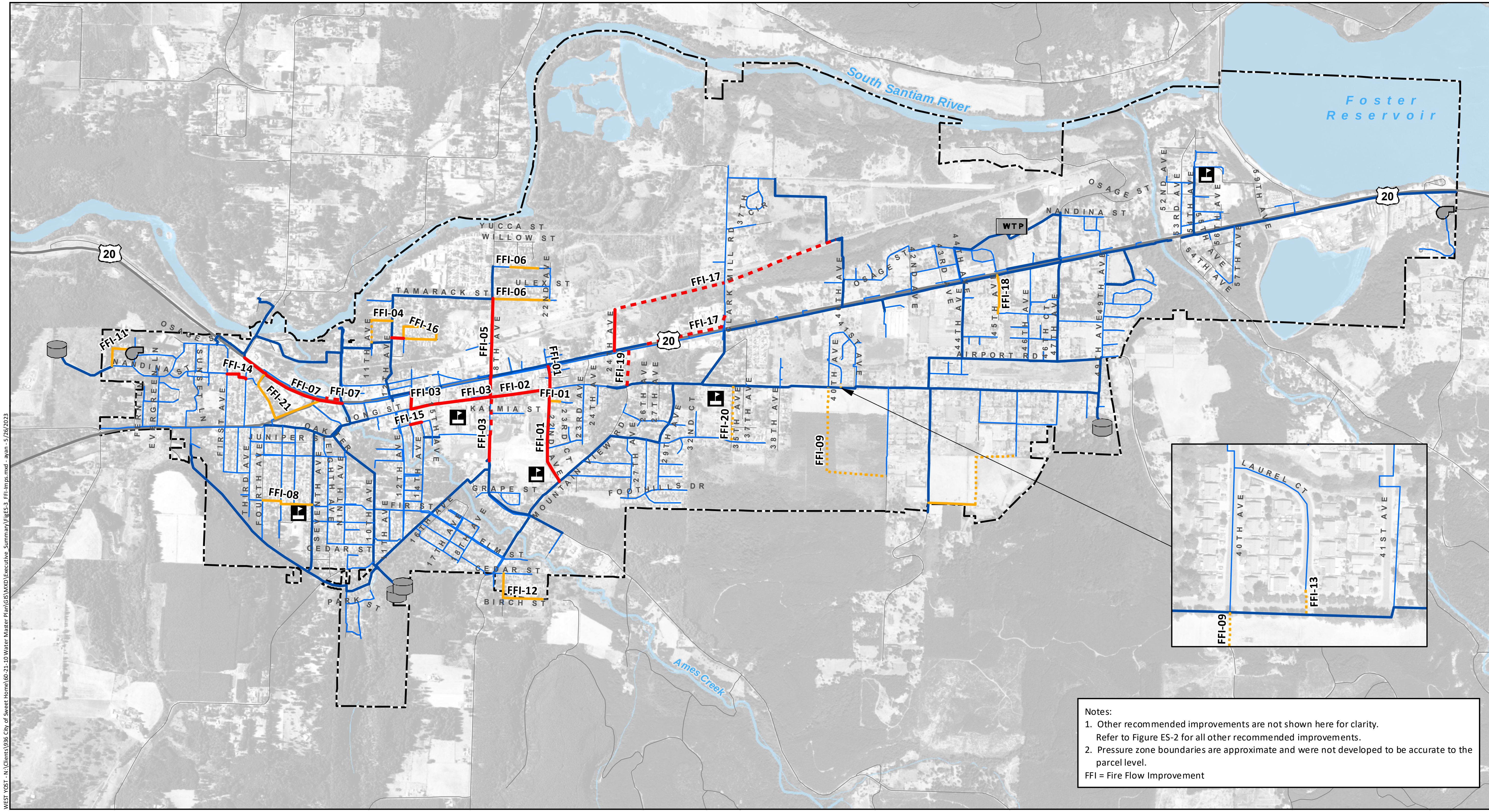
0 900 1,800

Scale in Feet

WEST YOST

Figure ES-2
Future System Recommended Non-Fire Flow Improvements

City of Sweet Home
Water Master Plan



WTP Existing Water Treatment Plant

Existing Storage Tank

Potable Water Pump Station

Existing System Pipelines

Diameter Less than 10-inches

Diameter 10-inches and Greater

Recommended Pipeline Improvement

FFI - New 8-inch Pipeline

FFI - New 12-inch Pipeline

FFI - Replace with 8-inch or 10-inch

FFI - Replace with 12-inch

City Limit

School

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N

0 900 1,800

Scale in Feet

WEST YOST

Figure ES-3
Future System Recommended Fire Flow Improvements

City of Sweet Home
Water Master Plan

SEISMIC RISK ASSESSMENT AND MITIGATION PLAN (CHAPTER 8)

The seismic resiliency assessment evaluates the seismic hazards present within the City of Sweet Home's (City) water service area and identifies their potential impacts to the water system after a major seismic event. A 9.0 Cascadia Subduction Zone (CSZ) earthquake was selected for the earthquake hazards analysis, consistent with the State of Oregon's 2013 Oregon Resilience Plan, which presents target states of recovery following a major earthquake and suggests planning for long-term goals for water system readiness in case of a magnitude 9.0 CSZ earthquake.

McMillen Jacobs Associates was contracted to complete a geotechnical seismic hazards evaluation of the City's service area. ACE Engineering LLC (ACE) was contracted to complete a structural seismic evaluation of the existing critical water structures in the water treatment and distribution system of the City. The results of the geotechnical and structural analyses indicate that the majority of the City's service area is not located within a seismic hazard zone and most of the critical water facilities are in reasonable structural condition.

The City's critical water system facilities were evaluated for seismic resiliency and the following mitigation strategies were developed for improving the seismic resiliency of the backbone water system:

- Pipe replacement: Replace existing Cast Iron (CI) pipes with more seismic resilient pipeline systems.
- Site-specific slope stability analyses are recommended to be performed at the 10th Avenue and 49th Avenue Reservoir sites to determine the level of seismic landslide hazard.
- Maintenance and structural upgrades should be part of the City's operating plan.
- Emergency training and exercises: Emergency training and exercises focused on earthquake scenarios can be implemented to enhance the City's emergency preparedness.

CAPITAL IMPROVEMENT PROGRAM (CHAPTER 9)

The recommended water system 5-year Capital Improvement Plan (CIP) and 20-year CIP are presented in Table ES-2, with an estimated capital cost of $10.6 Million (M) and $47.3M, respectively. The total overall CIP capital cost is approximately $57.9M as shown in Table ES-2. The recommended capacity and reliability, fire flow and small diameter mains improvement projects all will improve water system capacity and performance. Implementation of the water treatment plant improvements and seismic resiliency improvements will improve water system reliability and resiliency.

Table ES-2. Summary of Recommended Capital Improvement Projects[a]

Improvement Category	Improvement Reason	5-Year CIP Capital Cost, dollars	20-Year CIP Capital Cost, dollars	Total CIP Capital Cost, dollars
Operations and Maintenance				
Operations and Maintenance	• Conduct Operations and maintenance projects at the WTP as described in Chapter 7 • Address the non-structural considerations for each critical water facility as described in Chapter 8	-	-	$90,000
	Annual Operations and Maintenance Total	**-**	**-**	**$90,000**
Capital Improvements				
Capacity or Reliability Improvements	• Construct proposed improvements to meet performance criteria and long-term operational goals identified by the City, including the replacement of existing pipelines and the construction of new pipelines, pump stations, reservoirs, and PRVs	6,208,000	29,704,000	35,912,000
Fire Flow Improvements	• Construct proposed improvements to meet fire flow performance criteria,. including the replacement of existing pipelines and the construction of new pipelines	2,597,000	10,965,000	13,562,000
Small Diameter Mains Improvements	• Replace all City owned pipelines 2-inches in diameter	-	6,274,000	6,274,000
Seismic Improvements	• Implement mitigation strategies for improving the seismic resiliency of the backbone water system	-	310,000	310,000
Water Treatment Plant Improvements	• Address deficiencies in the treatment process identified from the condition assessment of the WTP	1,844,000	-	1,844,000
	Capital Improvements Total	**$10,649,000**	**$47,253,000**	**$57,902,000**

(a) Costs are rounded to the nearest thousand dollars. Improvements in this table are considered "backbone" improvements. Smaller, in-tract, improvements are not included and are assumed to be constructed by future development proponents. Costs are based on the May 2023 Engineering News Record Construction Cost Index (ENR CCI) of 13,288 (20-Cities Average).

CHAPTER 1
Introduction

1.1 WATER MASTER PLAN PURPOSE

The purpose of this Water Master Plan (WMP) for the City of Sweet Home (City) is to formulate a comprehensive, current Capital Improvement Program (CIP) that can serve as a roadmap to meet the needs of the City's existing and future water customers. In 2016, the City completed a combined Water Management and Conservation Plan and WMP. Since the City's previous WMP was developed, the City has implemented many of the recommended CIP projects and has completed significant water system improvement projects throughout the system. Therefore, this WMP serves to evaluate the current water system under existing and future demand conditions, identify any existing system deficiencies, and recommend water system improvements. Evaluations were based on updated demand estimates.

Evaluations and recommendations presented in this WMP are based on information collected in 2021 and 2022, including historical data and records, record drawings, past surveys and reports, current Geographic Information System (GIS), and results from requested field inspections/data collection collected for this WMP. The date range for each data type is specified when described in the chapters of this WMP. Updates and improvements completed within the City's water system through 2022 have been incorporated as part of this WMP.

1.2 WATER MASTER PLAN OBJECTIVES

The objectives of this WMP are to:

- Evaluate historical water meter data to develop current and estimated future water system average and peak demands;
- Identify design, operational, and performance criteria to guide the water system evaluations;
- Update the City's GIS-based water system hydraulic model and re-allocate recent demands to the hydraulic model;
- Analyze the existing distribution system to evaluate the ability of the City's water system to meet current and future demands using the water system hydraulic model;
- Evaluate the existing WTP for hydraulic capacity and to identify operations and maintenance (O&M) needs;
- Prepare a seismic resiliency analysis to evaluate seismic hazards and their potential impact on the water system;
- Identify system deficiencies and recommend upgrades to meet operational and performance criteria; and,
- Develop a comprehensive CIP to address existing system deficiencies.

1.3 AUTHORIZATION

West Yost was authorized to prepare this WMP by the City on September 2, 2021.

1.4 REPORT ORGANIZATION

This WMP is organized into the following chapters:

- Chapter 1: Introduction
- Chapter 2: Existing System Description
- Chapter 3: Water Demand
- Chapter 4: Design and Performance Criteria
- Chapter 5: Hydraulic Model Update
- Chapter 6: Water System Analysis
- Chapter 7: Water Treatment Plant Evaluation and Upgrades
- Chapter 8: Seismic Risk Assessment and Mitigation Plan
- Chapter 9: Capital Improvement Program

The following appendices to this WMP contain additional technical information, assumptions, and calculations:

- Appendix A: Hydrant Testing Plan
- Appendix B: Geotechnical Seismic Risks and Hazards Mapping
- Appendix C: Structural Seismic Resiliency Evaluation

1.5 ACKNOWLEDGMENTS

The development of this WMP would not have been possible without key involvement and assistance of the City's Public Works staff. In particular, the following staff provided comprehensive information, input, and insights throughout the development of the WMP:

- Greg Springman, Public Works Director, City of Sweet Home
- Dominic Valloni, Public Works Operations Manager, City of Sweet Home
- Steven Haney, Utilities Manager, City of Sweet Home
- Patricia Rice, Engineering Technician II, City of Sweet Home

CHAPTER 2
Existing System Description

This chapter describes the City's existing water distribution system. Water system information was obtained through review of previous reports, maps, plans, operating records, and other available data provided to West Yost by the City. The following sections of this chapter describe the key components of the City's existing water system:

- Existing Water Service Area
- Existing Water Supplies
- Existing Water System
- Existing Operations and Maintenance Programs

2.1 EXISTING WATER SERVICE AREA

The City is located within Linn County (County), Oregon, about 75 miles south of Portland, 40 miles southeast of Salem, and 30 miles northeast of Eugene. The City is situated in the foothills of the Cascade Mountain Range, in the eastern portion of the Willamette Valley. The City is bounded by the South Santiam River to the north, Foster Reservoir to the east, forested hills to the south, and primarily agricultural land to the west. United States (US) Route 20, the Santiam Highway, runs in an east-west direction and roughly bisects the City.

Figure 2-1 shows the City limit and the City's existing water service area. The existing water service area is approximately 3.65 square miles. The existing water service area consists of the County tax lots served by the City and generally falls within City limits. Elevations within the City limits range from approximately 850 feet mean sea level (msl) in the hills in the southern-most arm of the City to approximately 500 feet msl along the South Santiam River, where the river approaches the Santiam Highway on the west side of the City.

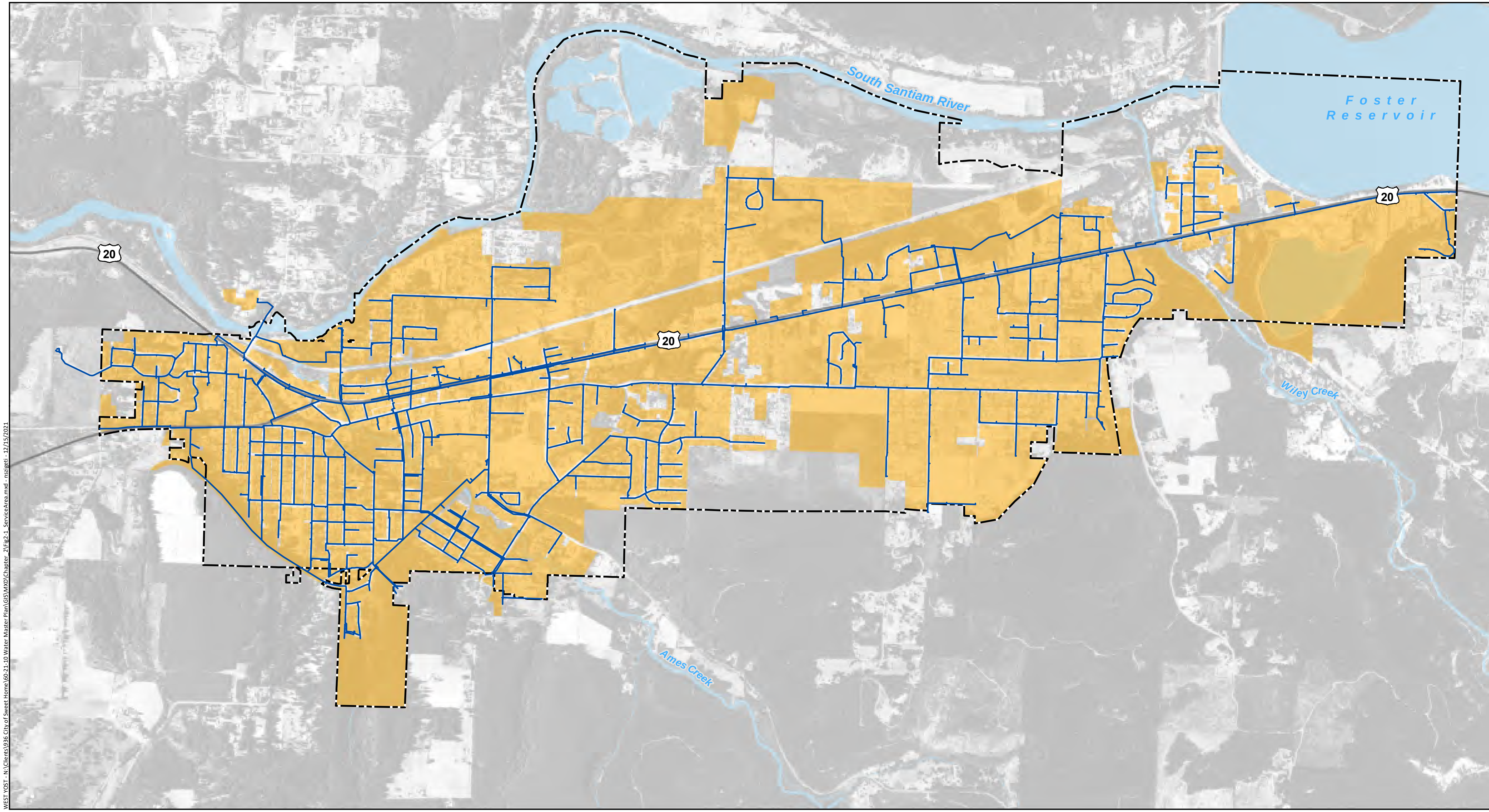
2.2 EXISTING WATER SUPPLIES

The City's existing water supply portfolio includes surface water from the South Santiam River, which is impounded at the Foster Reservoir, and Ames Creek. The following sections briefly describe these water sources and the City's drinking water quality and compliance history.

2.2.1 Sources of Water Supply

The City holds existing water rights to surface water from the South Santiam River and Ames Creek. Under Oregon law, water rights are obtained in a multi-step process. First, an applicant must apply to the Oregon Water Resources Department (ORWD) for a permit to use water. If the permit is approved, the permit holder must construct facilities to begin using water within a timeframe designated in the permit. The permit holder must hire a certified water right examiner to conduct a survey of the water use, also known as a "claim of beneficial use", which is submitted to ORWD for approval. If the water has been used according to provisions of the permit, ORWD will issue the permit holder a water right certificate. The certified or "perfected" water rights are based on the beneficial water use documented in the survey.

The following sections briefly describe these water sources and the City's water rights. Three (3) of the City's four (4) water rights are fully perfected. Therefore, the City's certified water rights are lower than the quantities identified in the water rights permits.



Water Service Area

City Limit

Pipelines

DRAFT

N

0 900 1,800

Scale in Feet

Figure 2-1

Existing Water Service Area

City of Sweet Home

Water Master Plan

2.2.1.1 South Santiam River

The City's primary water supply is surface water from the South Santiam River. As shown in Table 2-1, the City holds three existing water rights permits to the South Santiam River for municipal use that total 13.10 cubic feet per second (cfs), or approximately 8.47 million gallons per day (mgd). The City holds corresponding water rights certificates that total 11.11 cfs, or approximately 7.18 mgd. The difference in the quantities between the water rights permits and certificates is due to Permit Number (No.) S-49959, which is only partially perfected and has an associated certificate that is limited to 3.51 cfs (2.27 mgd). The City must demonstrate beneficial use of the remaining water right quantity of 1.99 cfs by October 1, 2050, to fully perfect Permit S-49959. Water rights Permit No. S-13151 and S-20525 are fully perfected.

The City diverts South Santiam River water from the Foster Reservoir through a 24-inch connection at the Foster Dam. The Foster Dam is a rock-fill dam constructed in 1968 and is owned and operated by the US Army Corps of Engineers (USACE). Figure 2-2 shows the location of Foster Reservoir, the raw water facilities, and water treatment plant (WTP).

2.2.1.2 Ames Creek

The City also holds certified water rights to Ames Creek, a tributary of the South Santiam River. Water Right No. 95551 allows the City to divert a maximum flow of 0.076 cfs (34 gallons per minute [gpm]) from Ames Creek for municipal use, as shown in Table 2-1. This certificate also limits the annual volume to 10 acre-feet (AF), or approximately 3.26 million gallons (MG). The City previously used this water right to serve municipal non-potable (i.e., irrigation) demands at the Sweet Home High School. At the time of this WMP the City does not divert water from Ames Creek.

2.2.1.3 Summary of Existing Water Rights

Table 2-1 summarizes the City's four existing water rights to the South Santiam River and Ames Creek.

Table 2-1. Summary of Existing Water Rights

Permit No.	Certificate No.	Point of Diversion	Priority Date	Permitted Water Right		Certified Water Right	
				cfs	mgd	cfs	mgd
S-13151	88300	South Santiam River	7/14/1938	0.60	0.39	0.60	0.39
S-20525	88301	South Santiam River	4/16/1951	7.00	4.52	7.00	4.52
S-49959	88302	South Santiam River	4/08/1986	5.50[(a)]	3.55	3.51	2.27
S-10140	95551	Ames Creek[(b)]	4/24/1931	0.076	0.049	0.076[(c)]	0.05
Total Available Water Right:				**13.18**	**8.52**	**11.19**	**7.23**
Total Available Water Right – Potable Use:				**13.10**	**8.47**	**11.11**	**7.18**

(a) Certificate No. 88302 is only partially perfected for 3.51 cfs of the 5.50 cfs under Permit No. S-49959. The City must apply the remaining 1.99 cfs to full beneficial use by October 1, 2050, to fully perfect the water right permit.
(b) Ames Creek surface water was previously used for non-potable irrigation at Sweet Home High School.
(c) Certificate No. 95551 limits the City to a maximum annual volume of 10 AF/yr (3.26 MG/yr) from Ames Creek.

2.2.2 Drinking Water Quality and Compliance History

The City fully treats its South Santiam River raw water supply for use as a municipal water supply per State and Federal regulations. The South Santiam River is considered a high-quality raw water source, as the upstream watershed largely consists of managed forestland with little development. The City has not experienced water quality or compliance issues since the new raw water pipeline, raw water pump station, and WTP were brought online in 2009. Water quality standards applicable to the City are described in detail in *Chapter 4 Design and Performance Criteria*.

2.3 EXISTING WATER SYSTEM

The City's key water system facilities are shown on Figure 2-2 and discussed in the sections below. Figure 2-2 shows a plan view of the City's distribution system and key water system facilities. The evaluation of facilities capacities and their ability to meet future water demands are described in *Chapter 6 Water System Analysis.*

2.3.1 Existing Water Treatment Facilities

The City's WTP receives and treats raw water from Foster Reservoir. The City's existing infrastructure used to convey and treat water for the potable distribution system is described in the sections below.

2.3.1.1 Foster Dam Raw Water Intake

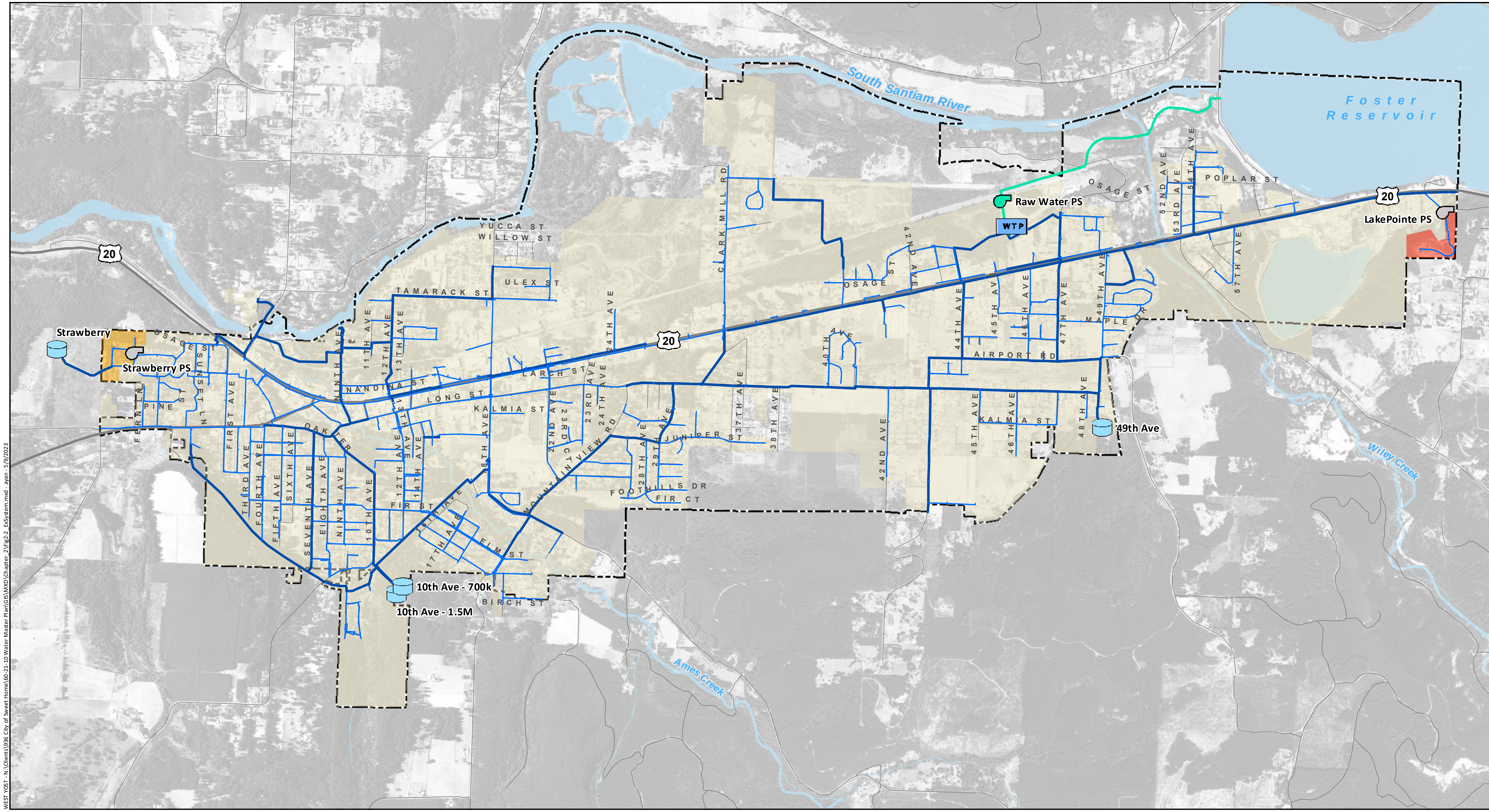
Foster Dam is owned and operated by the USACE. Foster Reservoir's low pool and full pool water surface elevations are 610 and 640 feet msl, respectively. Levels within the reservoir are maintained at the lowest elevations during winter months to allow for temporary storage of rainwater and snow melt, and the levels are gradually filled during the spring by the USACE to provide for recreation, water storage for municipal use, and downstream releases during the summer months.

The City diverts raw water from Foster Dam through a fish/debris screen and 24-inch connection at an elevation of 600 feet msl. A 24-inch ductile iron (DI) pipeline conveys raw water above-grade for approximately 600 feet before transitioning below-grade to a 30-inch high density polyethylene (HDPE) pipeline. This pipeline continues below-grade for approximately 4,600 feet, crossing Wiley Creek, and discharges into a raw water wet well with a maximum water surface elevation of 580.75 feet msl. The City pumps raw water from this wet well at an elevation of 572.75 feet msl to the water treatment plant using three raw water pumps. Each raw water pump is a 25 horsepower (hp) pump with a design capacity of 1,400 gpm at 50 feet of total dynamic head (TDH).

The City's existing raw water pipeline was constructed in 2007, and the raw water pump station was constructed in 2008.

2.3.1.2 Water Treatment Plant

The City's WTP was constructed in 2009. The City's water treatment facilities include a chemical feed system, static mixers, a tube clarifier, adsorption clarifier media, mixed media filter, and chemical disinfection. The treated and disinfected water then progresses through a 10-mgd baffled clearwell, where three finished water pumps (further described in Section 2.3.2.4) deliver the finished water to the City's water distribution system.



WTP Water Treatment Plant
Storage Tank
Potable Water Pump Station
Raw Water Pump Station

Raw Water Pipelines

Potable Water Pipelines

Diameter Less than 10-inches
Diameter 10-inches and Greater

City Limit

Pressure Zones

Main
Strawberry
LakePointe

Notes:

1. The finished water pump station is located on-site at the City's water treatment plant.
2. The 0.3 MG 10th Ave tank constructed in 1938 is currently offline and is not pictured.

N

0 900 1,800
Scale in Feet

WEST YOST

DRAFT

Figure 2-2

Existing Water System

City of Sweet Home
Water Master Plan

Each raw water pump feeds a single water treatment unit. The nominal capacity of each parallel system is 1,400 gpm, for a total WTP capacity of 4,200 gpm, or approximately 6.0 mgd. The firm capacity of the WTP is 2,800 gpm, or approximately 4.0 mgd, assuming one treatment system is out of service for maintenance or repair.

2.3.2 Existing Water Distribution System

The existing water distribution system includes:

- Pressure Zones
- Distribution Mains
- Storage Facilities
- Pump Stations

These systems are described below. The existing water distribution system is shown on Figure 2-2.

2.3.2.1 Pressure Zones

The City operates a total of three (3) pressure zones, as shown on Figure 2-2. The vast majority of the City's service connections are located in the Main Zone, which runs along Highway 20 from the east to west and serves all but the highest customer elevations. The finished water pump station at the WTP serves the Main Zone. The remaining two small pressure zones are supplied by booster pump stations pumping from the Main Zone as follows: the Strawberry Pump Station supplies the Strawberry zone and fills the Strawberry Reservoir; and the LakePointe Pump Station supplies the LakePointe Zone. Two connections locations above the 49th Avenue Reservoir are also served by a small pump station, though it is not maintained by the City and thus the area is not considered a City pressure zone. Zone-specific information is shown in Table 2-2.

Table 2-2. Summary of Existing Pressure Zones

Zone Name	Existing Minimum Service Elevation[a], feet	Existing Maximum Service Elevation[a], feet	Static Pressure Range, psi
Main	512	710	24 – 110[b]
Strawberry	655	736	35 – 71[b]
LakePointe	796	827	71 – 84[c]

(a) Service elevations are approximate based on 2009 bare earth Lidar data provided by City staff.
(b) Typical static pressure ranges were calculated from the tank overflow elevation associated with the corresponding zone from Table 2-5 minus the existing minimum and maximum service elevations associated with the corresponding zone.
(c) Typical static pressure range was calculated from the LakePointe Pump Station discharge pressure in the City's hydraulic model under average day demand conditions (0.85 mgd) minus the existing minimum and maximum service elevations within the LakePointe Zone.
psi = Pounds Per Square Inch

2.3.2.2 Distribution Mains

Table 2-3 and Table 2-4 summarize the City's existing pipelines by diameter and material type, respectively. The City's existing water system consists of approximately 54 miles of water system pipelines, with distribution pipelines sizes generally ranging from 2-inches to 8-inches in diameter. Transmission mains range from 10-inches to 24-inches in diameter, with 10-inch diameter pipelines comprising about 61 percent of the transmission mains. As shown in Table 2-3, approximately 50 percent (or 27 miles) of the City's pipelines are distribution mains consisting of pipelines 6 inches to 8 inches in diameter, while approximately 18 percent (or 10 miles) are small-diameter mains less than 6 inches in diameter. The City's predominant pipeline materials are DI (41 percent), polyvinyl chloride (PVC) (28 percent), or cast iron (CI) (21 percent).

Table 2-3. Summary of Existing Pipelines by Diameter

Pipe Diameter, inches	Length of Pipelines, feet	Length of Pipelines, miles	Percent of Water System
2	24,470	4.6	8.6%
3	6,149	1.2	2.1%
4	22,107	4.2	7.7%
6	64,203	12.2	22.4%
8	78,247	14.8	27.4%
10	55,451	10.5	19.4%
12	19,768	3.7	6.9%
16	15,266	2.9	5.3%
24	395	0.1	0.1%
Total	**286,056**	**54.2**	**100.0%**

Source: Potable water pipelines shapefile extracted from the City's hydraulic model, as of 11/30/2021.

Table 2-4. Summary of Existing Pipelines by Material

Pipe Material	Length of Pipelines, feet	Length of Pipelines, miles	Percent of Water System
Cast Iron (CI)	59,923	11.4	20.9%
Ductile Iron (DI)	116,137	22.0	40.6%
Galvanized Steel (GALV)	6,771	1.3	2.4%
Polyvinyl Chloride (PVC)	79,204	15.0	27.7%
Steel (STL)	4,990	0.9	1.7%
Unknown	19,031	3.6	6.7%
Total	**286,056**	**54.2**	**100.0%**

Source: Potable water pipelines shapefile extracted from the City's hydraulic model, as of 11/30/2021.

2.3.2.3 Storage Facilities

The City has five (5) storage reservoirs within its water service area, with a total storage capacity of 4.61 MG. At the time of this WMP, the oldest 10th Avenue reservoir (0.30 MG capacity) is offline due to leaks. Therefore, the total active storage capacity is 4.31 MG. The location of each reservoir is shown on Figure 2-2, with key information for each facility shown in Table 2-5. Storage reservoirs serving the Main and Strawberry Zones are each sited at an elevation that establishes the hydraulic grade for the pressure zone, which allows the reservoir to supply the zone by gravity. It should be noted that the Strawberry Reservoir has a large volume relative to the existing water demands in the Strawberry Zone, so the City actively monitors low chlorine residuals in the reservoir. Currently, chlorine residuals are maintained by continually running a metered faucet to increase reservoir turnover.

Table 2-5. Summary of Existing Potable Water Storage[a]

Facility Name	Pressure Zone	Diameter, feet	Construction Year	Construction Type	Base Elevation, feet	Overflow Elevation, feet	Nominal Storage Capacity, MG
10th Ave - 300K (Offline)	Main	64.0	1938	Partially Buried Concrete	749.5[b]	765.0[c]	0.30
10th Ave - 700K	Main	85.6	1951	Partially Buried Concrete	745.3[b]	765.0[c]	0.70
10th Ave - 1.5M	Main	105.0	1969	Partially Buried Concrete	742.0	765.0	1.50
49th Ave	Main	120.0	1993	Prestressed Reinforced Concrete	741.4	765.0	2.00
Strawberry	Strawberry	29.0	2001	Welded Steel	795.5	818.0[d]	0.11
						Total Capacity	**4.61**

(a) Where available, information was obtained from as-built construction records provided by City staff.
(b) The base elevations were estimated by subtracting the as-built maximum water height from the overflow elevation.
(c) Overflow elevations for the 1938 and 1951 reservoirs are not specified in the as-builts, and were approximated at 765 feet.
(d) Overflow elevation of the Strawberry reservoir is approximately 3 feet higher than indicated in the City's record drawings (815 feet), per City staff.

2.3.2.4 Pump Stations

The City currently operates three (3) pump stations within its water service area. The finished water pump station supplies the system from the WTP, and the remaining pump stations draw from the Main Zone to serve higher elevations within the system. Pump station locations are shown on Figure 2-2. The size and number of pumps varies at each pump station. Where multiple pump units are available, one pump is typically reserved as a standby unit. LakePointe Pump Station has backup power supplied by a natural gas generator, and there is no backup power to the other pumps.

The total existing firm capacity, with the largest pump reserved as a standby unit at each pump station, is 3,750 gpm (5.4 mgd). Table 2-6 summarizes the key characteristics of the City's existing booster pump stations.

Table 2-6. Summary of Existing Potable Water Pumps[(a)]

Pumping Facility, Zone	Service Zone, Source Zone	Location	Pump ID/ Serial Number	hp	Design Flow, gpm	TDH, ft	Total Pumping Capacity, gpm	Firm Pumping Capacity, gpm
WTP Finished Water Pumps[(b)]	Main (WTP)	Water Treatment Plant	161886	100	1400	240	4,200	2,800
			161887	100	1400	240		
			161888	100	1400	240		
Strawberry Booster Pump Station	Strawberry (Main)	Between 525 and 497 Strawberry Loop	Unknown	5	100	65	200	100
			Unknown	5	100	65		
LakePointe Booster Pump Station[(c)]	LakePointe (Main)	1200 Riggs Hill Road	Unknown	15	100	246	1,500	850
			Unknown	15	100	246		
			Unknown	40	650	187		
			Unknown	40	650	187		
						Total	**5,900**	**3,750**

(a) Information based on as-built construction documents and manufacturer design information provided by City staff.
(b) WTP finished water pumps are part of the WTP and draw suction directly from the clearwell.
(c) The LakePointe pumps are equipped with variable frequency drive (VFD) motors.
hp = Horsepower

2.4 WATER DISTRIBUTION SYSTEM OPERATIONS AND MAINTENANCE

2.4.1 Organizational Structure

The City's Public Works department is organized as illustrated on Figure 2-3. The City's water treatment and distribution system is operated by two WTP operators, a water distribution and collections systems crew leader, and three distribution system maintenance workers. The Utilities Manager, Engineering Technician II, and Operations Manager oversee the planning, engineering, and construction of new water system facilities, and provide general oversight of the City's water system and operations and maintenance activities. Four seasonal temporary maintenance workers are also on staff, one for each branch of the City's Public Works department.

As of the preparation of this WMP, the City has identified the WTP operator position as an underfilled role. Other underfilled roles within the Public Works Department that do not directly pertain to the water system are not listed here.

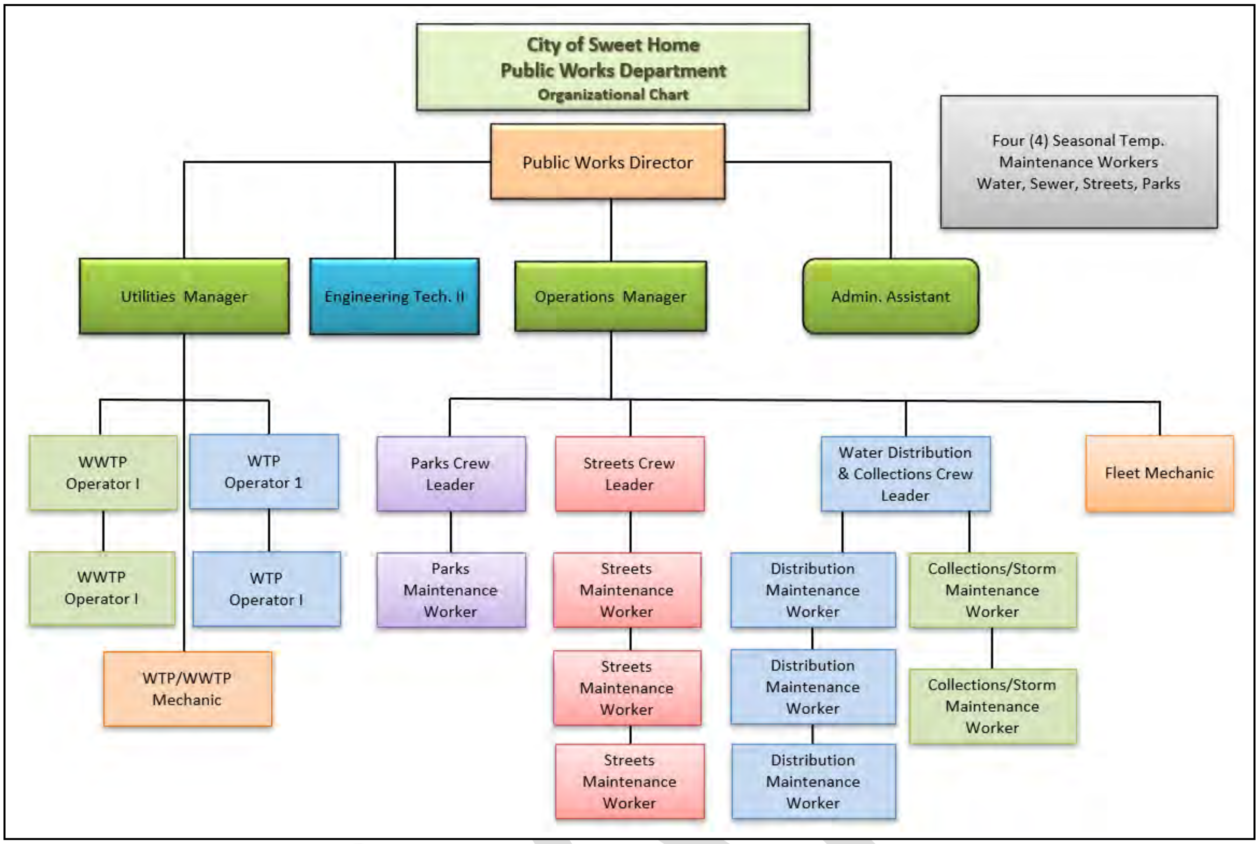


Figure 2-3. City Public Works Organizational Chart

2.4.2 Existing Distribution System Operations and Maintenance Programs

The City performs several Operation and Maintenance (O&M) programs which aim to extend the useful life of its assets, identify deficiencies, and upgrade aging infrastructure. These programs are summarized as follows:

- **Hydrant Flushing Program:** The City flushes hydrants quarterly or annually, dependent on location, to improve water quality.
- **Leak Detection Program:** The City proactively identifies and fixes leaks via acoustic leak detection.
- **Hydrant Testing and Maintenance Program:** The City tests hydrants every three years and services hydrants as needed.
- **Valve Exercising Program:** The City operates its main valves every five years to extend the useful life and track the condition of the City's valves.
- **Meter Replacement Program:** The City replaces mechanical water meters monthly in an ongoing effort to convert the entire system to ultrasonic meters. While this has taken place for over ten years, the City plans to complete the program in 2022.
- **Regulatory Water Quality Testing:** The City regularly tests water quality at specific locations throughout the service area to demonstrate compliance with state and federal regulations. These regulations are described in detail in *Chapter 4 Design and Performance Criteria.*

In addition to the programs listed above, the City actively works to improve its water system operations and reliability through implementing new O&M programs on an as-needed basis. New programs that have recently been implemented or have been identified for administration in the near-term include:

- **Meter Reading:** The City has recently brought meter reading responsibilities in-house.
- **Bridge Inspection Program:** City staff are developing a routine bridge crossing inspection plan. The City intends to conduct annual, proactive inspections of critical pipelines spanning bridge crossings to prevent pipeline main breaks along spans where leaks are historically difficult to detect leak.

As the City continues to invest in new and enhanced O&M programs to improve water system reliability, it is recommended that a periodic review of Operations staff workload be conducted. This review should evaluate whether existing City staff can reasonably complete all required O&M programs on recommended intervals, or whether the City should consider hiring an additional staff member to assist in meeting and maintaining the City's level of service goals.

CHAPTER 3
Water Demand

This chapter presents the current and projected potable water demands served by the City within its water service area. Accurate potable water demand estimates are necessary to develop and calibrate the potable water system hydraulic model, identify capacity deficiencies in the existing potable water system, and deliver a focused and comprehensive CIP. Future water demand projections are based on population growth within the service area and help the City identify and secure sufficient water supplies to serve their customers.

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

- Service Area Description
- Historical Water Production and Consumption
- Projected Water Demand

3.1 SERVICE AREA DESCRIPTION

The following subsections summarize characteristics of the City's existing water service area, including the existing service connections and the historical and projected population.

3.1.1 Existing Service Connections

The City tracks water services within its service area by billing class. For this WMP, the billing classes have been consolidated into six water use classes: Single Family Residential, Multi-Family Residential, Commercial, Industrial, Public Facilities, and Irrigation. There are approximately 3,200 water service connections in the City, of which 91 percent are Residential. Commercial connections account for approximately 6 percent, while Public Facilities connections account for approximately 3 percent. Table 3-1 provides a summary of the total water service connections by billing class.

3.1.2 Historical and Projected Population

As described in Chapter 2, the City's water service area is generally contiguous with the City limits. The City's current and forecasted population is estimated by the Portland State University (PSU) Population Research Center (PRC). The PRC produces annual certified population estimates for Oregon using U.S. Census data, an estimated natural increase (using State registration of births and deaths), and an estimated net migration (using data on school enrollment, employment, labor force, income tax exemptions, issued drivers licenses, voter registration, and Medicare enrollees). Population estimates for each city are developed using data on housing stock changes provided by City officials.

Approximately 9,400 people currently live in the City. As shown in Table 3-2, the PRC population estimates indicate that the City experienced an overall population growth of 3.1 percent from 2010 to 2018, equal to an annual growth rate of approximately 0.39 percent. From 2018 to 2020, the City's growth accelerated and its population increased 2.1 percent, increasing the annual growth rate to approximately 1.03 percent. Although 2020 U.S. Census results were made available during the preparation of this WMP, and are shown in Table 3-2, the PRC-estimated population of 9,415 for 2020 is used in this WMP to maintain a consistent approach across City planning documents.

According to the 2020 PSU PRC population forecast, the City's 2040 population is projected to increase to 11,010. However, future population estimates were developed for the City's *Wastewater Facilities Plan*, dated December 2016, using the 1.168 percent annual average growth rate (AAGR) predicted for Linn County, in accordance with OAR 660-032-0040(6), to project a 2040 population of 12,259. The draft *System Development Charge (SDC) Methodology Report*, dated December 2020, is consistent with the *Wastewater Facilities Plan* and assumes a 2040 population of 12,259. For the purposes of this WMP, the City's 2040 population projection consistent with other adopted planning documents is used. Therefore, the City's 2040 population is assumed to be 12,259. Population estimates presented for the five-year increments between 2020 and 2040 were interpolated assuming an average annual growth rate of 1.3 percent per year. Finally, as this WMP encompasses a 20-year planning horizon, the 2043 (future) population was extrapolated using the average annual growth rate of 1.3 percent per year. Table 3-2 presents the City's projected future population of 12,758.

Table 3-1. Existing (2020) Service Connections

Service Use Class	Service Billing Class	Number of Connections[a]
Single Family Residential	Residential	2,824
Multi-Family Residential	Multi-Family	74
Commercial	Commercial	12
	Commercial -High	26
	Commercial-Low	117
	Commercial-Medium	17
	Hotel/Motel	3
Industrial	Industrial	10
Public Facilities	Church/Meeting Halls	26
	Federal	8
	Medical	6
	Municipal	34
	School	8
	State	1
Irrigation/Fire	Fire	11
	Irrigation/Fire	14
	Total	**3,191**

(a) Based on December 2020 billing records provided by the City.

Table 3-2. Historical and Projected Population			
Year	PSU PRC Estimates[(a)]	City Planning Documents[(b)]	US Census[(c)]
Historical Population			
2010	8,945	--	8,925
2011	9,005	--	--
2012	9,025	--	--
2013	9,065	--	--
2014	9,060	9,060	--
2015	9,090	--	--
2016	9,090	--	--
2017	9,090	--	--
2018	9,225	--	--
2019	9,340	9,340	--
2020	9,415	--	9,828
Projected Population			
2025	10,046	10,058	--
2030	10,455	10,745	--
2035	10,759	11,479	--
2040	11,010	12,259	--
2043	--	12,758	--

(a) Yearly estimates obtained from the 2020 Annual Oregon Population Report Tables, PSU PRC, revised July 1, 2020. Projected population obtained from the Current Forecast Summaries for All Areas, revised 2021.

(b) The City used a 20-year future population of 12,259 people in its 2020 SDC Methodology Report, consistent with the Wastewater Facilities Plan. Five-year incremental future population estimates shown in Table 3-2 were linearly interpolated between the 2020 PSU PRC population estimate (9,415) and the 2040 future population in other City planning documents (12,259).

(c) United States Census Population Estimates. April 1, 2020.

3.2 HISTORICAL WATER PRODUCTION AND CONSUMPTION

The City utilizes surface water from Foster Reservoir as the primary potable water source and treats it at the City's water treatment plant before distributing it to the water system. Water production is the quantity of water treated and distributed to the water system for customer use. Water consumption is equal to the metered water use. The difference between production and consumption is non-revenue water (NRW).

The following subsections detail the City's historical production and consumption (including per capita use), NRW, and peaking factors reflecting the seasonal variation in demands.

3.2.1 Water Production

Table 3-3 summarizes the City's historical annual water production from 2016 through 2020. Actual water production dropped approximately 20 percent in 2020 from the average (2016 to 2019) annual production of 436.5 MG. The decrease in 2020 annual production can be attributed to water savings experienced after the City fixed a large water leak in April 2020, which was located in 9th Avenue near the old water treatment plant. The leak was estimated to consistently account for approximately

343,000 gallons per day (gpd), beginning in 2012. Because this leak accounted for approximately 30 percent of the actual average day production prior to 2020, the daily production was adjusted (decreased by 343,000 gpd) for planning purposes to capture historical production trends, assuming no leak in the system. The adjusted production is presented with the actual production in Table 3-3 and shown in Figure 3-1.

Table 3-3. Historical Annual Water Production

Year	Total Production, MG		Average Day Production, mgd	
	Actual[a]	Adjusted[b]	Actual[a]	Adjusted[b]
2016	418.3	292.8	1.14	0.80
2017	436.1	310.9	1.19	0.85
2018	451.2	326.0	1.23	0.89
2019	440.5	315.3	1.20	0.86
2020	345.9	309.5	0.95	0.85
Average	**418.4**	**310.9**	**1.15**	**0.85**

(a) Daily production data provided by the City for 2016 through 2020.
(b) To account for a large water leak, 0.343 mgd was subtracted from the daily measured production through April 15, 2020. Actual production after the leak was repaired in April 2020 is assumed to be representative of water use and was not adjusted.

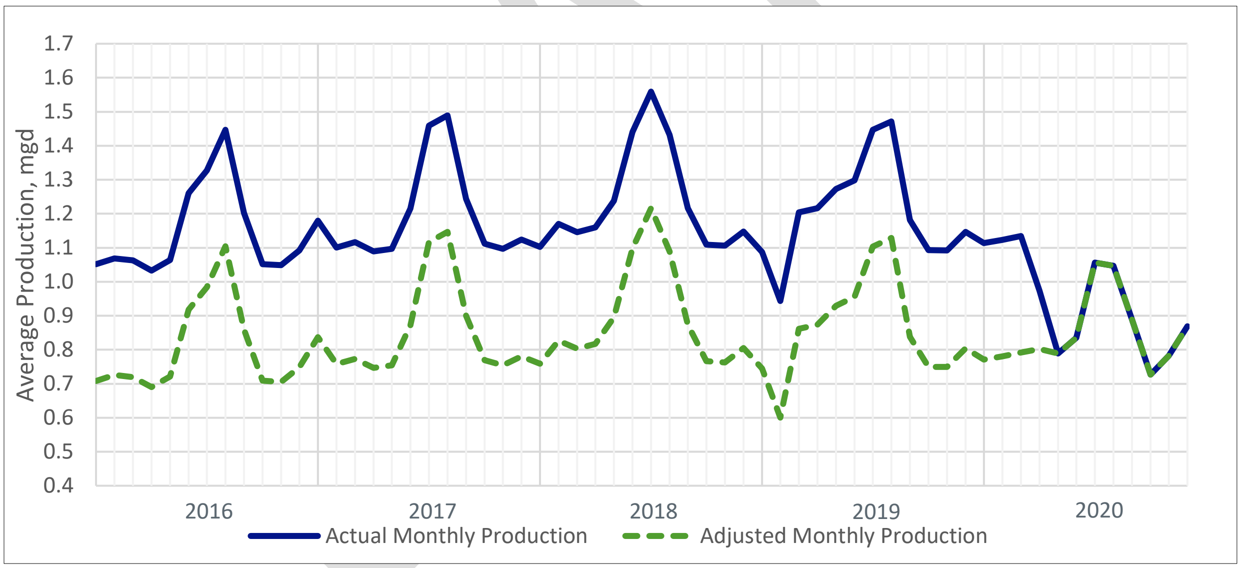


Figure 3-1. Monthly Production

3.2.2 Water Consumption

Table 3-4 presents the City's historical annual water consumption by service use class from 2016 to 2020. Single family residential and industrial water consumption have increased over the last five years, while all other water use has decreased.

Table 3-4. Historical Metered Water Consumption

Service Use Class	Annual Consumption, MG				
	2016	2017	2018	2019	2020
Single Family Residential	142.2	148.5	127.3[a]	141.0	157.2
Multi-Family Residential	23.6	25.8	44.0[a]	22.0	20.4
Commercial	18.7	19.5	17.4	16.7	15.1
Industrial	1.1	1.1	0.9	1.0	1.3
Public Facilities	38.6	37.6	32.7	38.4	35.6
Irrigation/Fire	8.1	7.0	8.9	8.3	6.1
Total, MG	**232.3**	**239.5**	**231.2**	**227.4**	**235.7**
Total, mgd	**0.63**	**0.66**	**0.63**	**0.62**	**0.64**

Source: City of Sweet Home billing information, received 12/14/2021.

(a) Both single family water use and multi-family metered water use are outliers in 2018 compared to other years on record. Some single-family accounts may have been misclassified as multi-family accounts for this year only.

The City's largest water user is the City wastewater treatment plant (WWTP). From 2016 to 2020, the WWTP accounted for approximately 7 percent to 9 percent of the City's total annual metered consumption, as shown in Table 3-5. The WWTP uses potable water for process water. Process water use is generally consistent throughout the year and does not exhibit daily or seasonal use patterns. Current improvements at the WWTP will replace the potable water used for process water with finished water produced on-site. This improvement will reduce the future potable water consumed by the WWTP. For planning purposes, it is assumed that the potable water demand for process water at the WWTP will remain consistent with observed water use, or approximately 19 MG annually.

Table 3-5. Historical Wastewater Treatment Plant Process Water Consumption

Year	Total Metered Consumption[a], MG	WWTP[b]	
		Annual Consumption, MG	Percent of Total Metered Consumption
2016	232.3	21.3	9.2%
2017	239.5	19.8	8.3%
2018	231.2	16.6	7.2%
2019	227.4	18.0	7.9%
2020	235.7	20.0	8.5%
Average	**233.2**	**19.1**	**8.2%**

Source: City of Sweet Home billing information, received 12/14/2021.

(a) Refer to Table 3-4.
(b) WWTP demand based on billing records for account number 004679-000.

The City also uses potable water to backwash the filters at the WTP. Existing finished water pumps at the WTP pump potable water into the distribution system. A flow meter records the total produced water entering the system (i.e., a flow totalizer). Under current operating conditions, backwashing the filters requires drawing potable water directly from the distribution system to use system pressure to reverse flow through the filters. Since the backwash supply line is located between the finished water pumps and the flow meter, backwashing requires drawing potable water through the flow meter in reverse. The flow totalizer does not measure the reverse flow through the meter so the potable water used for backwash is measured manually using a separate meter on the backwash pipeline. From 2016 to 2020, backwashing at the WTP accounted for approximately 2 percent to 7 percent of the City's total annual production as shown in Table 3-6. A capital project to install a pump to backwash the filters with water from the clearwell is currently in construction which will eliminate the need to use potable water for backwashing. For planning purposes, it is assumed that backwash at the WTP will not contribute to potable water demand in the future.

Table 3-6. Water Treatment Plant Backwash Water Usage

Year	Total Adjusted Production[a], MG	WTP[b]	
		Total Backwash Usage[b], MG	Percent of Total Adjusted Production
2016	292.8	6.6	2.3%
2017	310.9	14.6	4.7%
2018	326.0	13.2	4.0%
2019	315.3	22.7	7.2%
2020	309.5	16.6	5.4%
Average	**310.9**	**14.7**	**4.7%**

Source: City of Sweet Home WTP backwash data, received 7/15/2022.

(a) Refer to Table 3-3.
(b) WTP backwash meter reads provided by City Staff.

3.2.3 Non-Revenue Water

NRW is the difference between the quantity of water produced and the quantity of water consumed or metered. Customer water use typically does not equal the total water production because of system losses. These "lost" flows, previously referred to as unaccounted-for water, are now referred to as NRW. In 2003, the American Water Works Association (AWWA) abandoned use of the term "unaccounted-for water."[1] All water supplied to a distribution system can be accounted for, either as beneficial consumption, real losses (such as pipeline leakage), or apparent losses (such as measurement error). Therefore, the term NRW is favored to quantify water loss.

AWWA specifically defines NRW to include specific types of water loss, including any authorized, unbilled consumption (e.g., backwashing the WTP filters, flushing, etc.). However, for the purposes of this WMP, the NRW will not include metered consumption that is authorized but unbilled (i.e., WWTP process water and WTP backwash water). The City's NRW may consist of pipeline leakage, hydrant flushing, water used

[1] Best Practice in Water Loss Control: Improved Concepts for 21st Century Water Management, AWWA (2016).

for fire fighting, leaky meters, large fluctuations in the reservoir levels, and/or other real or apparent losses.

In recent years, the City has made a concerted effort to reduce NRW with the following actions:

- **Water Meter Replacement:** The City is currently replacing all customer water meters with a target completion date in 2022. Existing customer water meters are old, prone to leaks, and do not read low flows (e.g., a slow leak, such as a leaky toilet, can go undetected). Water meters are being replaced with ultrasonic meters that will be more accurate at lower flows.
- **Leak Detection:** The City maintains a large inventory of distribution system pipelines relative to its population and overall water demand, which increases the system's potential for leaks. Traditionally, the City addressed water leaks on an as-needed basis. A few years prior to this WMP, the City hired a leak detection company to conduct a pilot leak detection program. Based on the success from the pilot program, the City has invested in a proactive approach and has incorporated leak detection into its routine operations and maintenance.

Table 3-7 summarizes the City's NRW from 2016 through 2020. As described previously, the City's total production was adjusted to account for the approximate 343,000 gpd leak that was fixed in April 2020. Therefore, NRW is calculated as the adjusted total production less the metered consumption, including the WWTP process water, and the metered WTP backwash. For planning purposes, an average NRW of approximately 20 percent is recommended for use in future demand projections.

Table 3-7. Historical Non-Revenue Water

Year	Total Adjusted Production[(a)], MG	Total Consumption[(b)], MG	Total WTP Backwash[(c)], MG	Water Loss[(d)], MG	Non-Revenue Water[(e)], %
2016	292.8	232.3	6.6	53.9	18.4%
2017	310.9	239.5	14.6	56.8	18.3%
2018	326.0	231.2	13.2	81.6	25.0%
2019	315.3	227.4	22.7	65.2	20.7%
2020	309.5	235.7	16.6	57.3	18.5%
Average	**310.9**	**233.2**	**14.7**	**63.0**	**20.2%**

(a) Total Adjusted Production used to calculate NRW accounts for water losses attributed to the large leak repaired in April 2020. Refer to Table 3-3.
(b) Refer to Table 3-4.
(c) Refer to Table 3-6.
(d) Water Loss is calculated as the Total Adjusted Production minus the Total Consumption and Total WTP Backwash.
(e) NRW is calculated as Water Loss divided by the Total Adjusted Production. For the purposes of this WMP, the NRW will not include metered consumption that is authorized but unbilled (i.e., WWTP process water and WTP backwash water)

An estimate of NRW is required for water system planning to project future water production needs, as a system will always contain some amount of water loss. Water providers strive to minimize the amount of NRW, but it is difficult to eliminate entirely. A NRW percentage of 20 percent is on the high end of many water utilities but would not be considered excessive or indicative of a major problem in the City's water distribution system. A high NRW can be experienced in water systems where the overall demands are small and any routine maintenance (i.e., hydrant testing, flushing, or tank maintenance) could have a significant impact on the overall percentage of NRW. A high NRW can also been seen in water systems that experience a large volume of water lost to leaks. Since the City maintains a large inventory of distribution system pipelines relative to its population and overall water demand, its potential for leaks may be higher than the potential for leaks at a water utility with fewer miles of pipeline but which serves a similar customer population and/or volume of water. In addition, real losses exert a larger proportional impact on a system with low customer demands.

3.2.4 Per Capita Water Use

Table 3-8 summarizes the City's historical per capita water use from 2016 to 2020 based on the PSU PRC population estimates discussed in Section 3.1.2. Per capita water use is used to estimate the City's future water use as its population increases, assuming the relative distribution of residential and non-residential land uses are not anticipated to change appreciably. Since the WWTP process water is anticipated to remain constant and improvements to the WTP will reduce the potable water consumed for filter backwashing, Table 3-8 presents the net water production serving customers in the distribution system. For planning purposes, the total net water production was assumed to be the adjusted total production (from Table 3-3) minus the WTP filter backwash (from Table 3-6) and the WWTP process water (from Table 3-5). It is recommended that City's average per capita water use of 82 gallons per capita per day (gpcd) be used for projecting future water use in the City's service area.

Table 3-8. Summary of Per Capita Water Use

Year	Population[a]	Net Water Production[b], MG	Per Capita Water Use, gpcd
2016	9,090	264.9	79.6
2017	9,090	276.5	83.3
2018	9,225	296.2	88.0
2019	9,340	274.6	80.5
2020	9,415	272.9	79.2
Average	**9,232**	**277.0**	**82.1**

(a) PSU PRC population estimates are presented in Table 3-2.
(b) Per discussion with City Staff, net water production attributed to customer water use has been calculated as the Adjusted Production (Table 3-3) minus WWTP process water usage (Table 3-5) minus backwash water usage (Table 3-6).

3.2.5 Peaking Factors

Accurate peak demands are critical for evaluating and sizing water system transmission/distribution pipelines and storage facilities and defining water supply needs and capacity requirements. Projecting peak demands typically involves applying a multiplier, or peaking factor, to the average day demand. An average day demand for a particular year is calculated by taking the total annual water production divided by the total number of days in that year (refer to Table 3-3).

Historical water use data help identify appropriate peaking factors for key demand conditions. The following subsections describes the methodology used to develop the City's maximum day demand (MDD) and peak hour demand (PHD) peaking factors.

3.2.5.1 Maximum Day Demand Peaking Factor

The MDD peaking factor is calculated by dividing the calendar year's largest, single-day demand by the average day demand (ADD) of the same year.

The maximum day peaking factors were calculated using the net water production, as described in Section 3.2.4. Due to planned improvements, WTP backwash water was assumed to not contribute to potable water demands and were excluded from both the average day and maximum day production. Furthermore, WWTP process water was assumed to not vary seasonally (i.e., a MDD peaking factor of 1.0 times the ADD) and has been excluded from the historical MDD peaking factor calculations. Based on these assumptions, Table 3-9 presents the maximum day peaking factors from 2016 through 2020. The maximum day peaking factor ranged from 1.7 (2018) to 2.9 (2019), with an average of 2.4. For planning purposes, a MDD peaking factor of 2.4 times the ADD is recommended.

Table 3-9. Historical Maximum Day Demand Peaking Factors

Year	Average Day Net Production,[a] mgd	Historical Maximum Day					MDD Peaking Factor
		Date	Total Adjusted Production,[b] mgd	WWTP Process Water,[c] mgd	WTP BW Water,[d] mgd	Maximum Day Net Production,[e] mgd	
2016	0.73	August 14	1.91	0.06	0.00	1.85	2.56
2017	0.76	October 27	1.86	0.05	0.11	1.69	2.23
2018	0.82	July 13	1.44	0.05	0.00	1.39	1.72
2019	0.76	May 19	2.26	0.05	0.04	2.16	2.87
2020	0.75	July 30	1.84	0.05	0.00	1.79	2.40
Average	**0.76**	**-**	**1.86**	**0.05**	**0.03**	**1.78**	**2.36**

(a) Refer to Net Water Production values in Table 3-8.
(b) Measured maximum day production values were adjusted to account for a water leak equal to 343,000 gpd through April 15, 2020.
(c) Refer to Table 3-5. Process water is recorded monthly and could not be determined on the maximum day, therefore, the annual average was used.
(d) Maximum day backwash meter reads provided by City Staff on 7/15/2022.
(e) Maximum day net production = Total Adjusted Production – WTP BW Water –WWTP Process Water.

3.2.5.2 Peak Hour Demand Peaking Factor

The PHD peaking factor is typically calculated by dividing the calendar year's largest single hour demand by the ADD of the same year. However, insufficient data was available to determine a historical peak hour demand factor. A review of other Western Oregon communities with similar climate and variation in seasonal demand indicates that a PHD of 1.5 times the MDD is appropriate for planning purposes. Therefore, a PHD peaking factor of 3.6 times the ADD is recommended.

3.2.5.3 Recommended Peaking Factors

The peaking factors presented in Table 3-10 are recommended for planning purposes.

Table 3-10. Recommended Maximum Day and Peak Hour Demand Peaking Factors	
Demand Condition	City
Average Day Demand	1.0 x ADD
Maximum Day Demand	2.4 x ADD
Peak Hour Demand	3.6 x ADD

3.3 PROJECTED WATER DEMAND

Future water demand projections for the City were developed using a population-based method, in which water demand is assumed to mirror population growth and residential and non-residential water use percentages are assumed to not significantly change. Projected water demands were calculated by multiplying the estimated future population by the per capita water use factor recommended in Section 3.2.4, and adding the average WWTP process water use from 2016 through 2020. Table 3-11 presents the projected water demand for City in five-year increments through 2043.

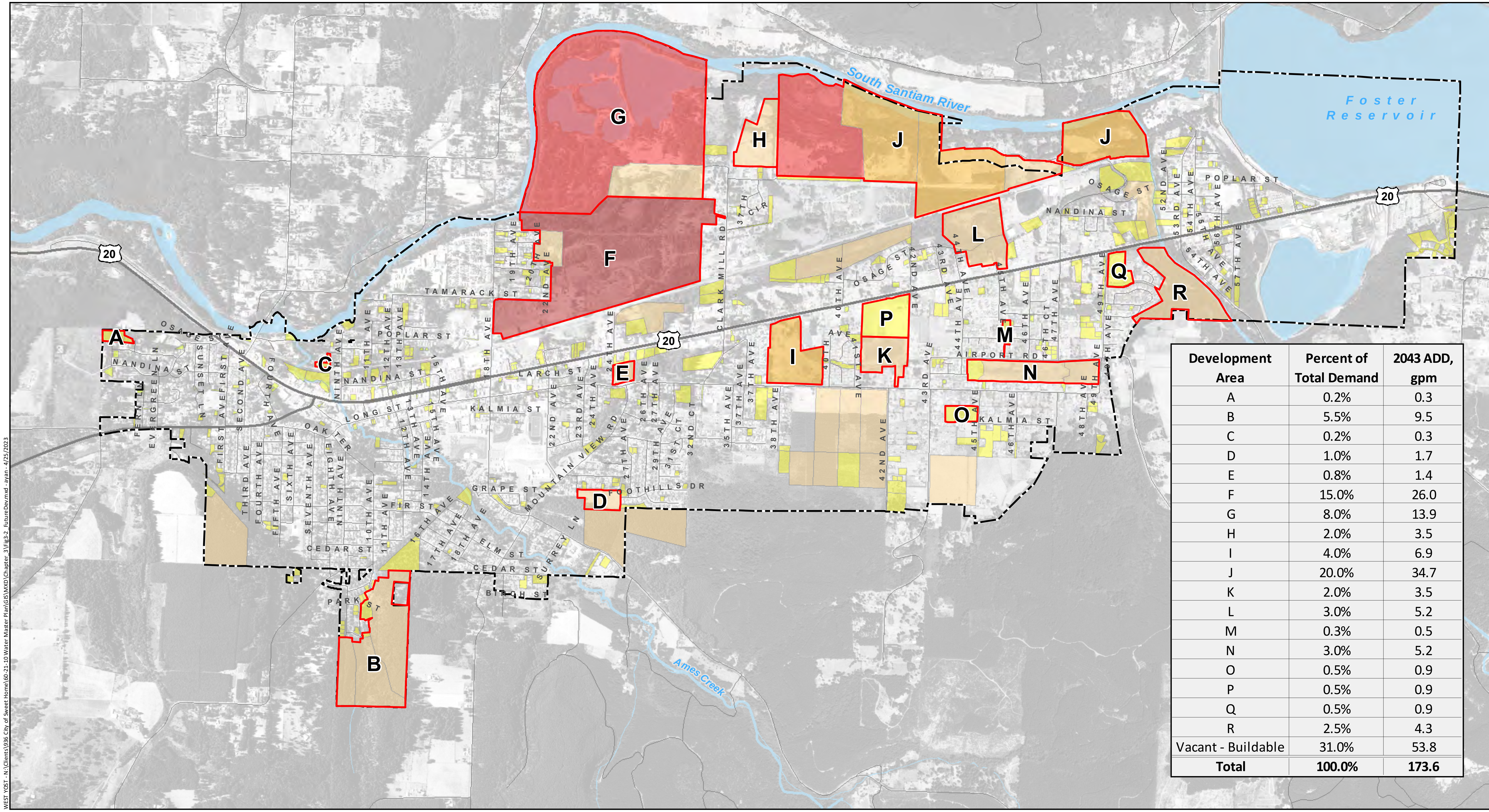
Table 3-11. Projected Water Demand[a]

Year	Projected Population[b]	Representative Per Capita Water Demand Factor,[c] gpcd	Required Daily WWTP Process Water,[d] mgd	Required Average Daily Water Production, mgd	Required Annual Water Production, MG
2025	10,058	82	0.05	0.87	317.6
2030	10,745			0.93	339.5
2035	11,479			0.99	361.4
2040	12,259			1.06	388.0
2043	12,758			1.10	401.5

(a) Includes non-revenue water.
(b) Refer to Table 3-2.
(c) Refer to Table 3-8.
(d) Refer to Table 3-5. The average annual WWTP process water use was used.

The City's average day water demand is projected to increase by approximately 0.25 mgd (176.3 gpm) by 2043 due to population growth. Figure 3-2 illustrates the distribution of new demand throughout the City. Known new developments were identified by the City via conference call on March 23, 2022 and are shown on Figure 3-2 as Development Areas A through G. Buildable vacant parcels were identified in GIS based on available tax lot information, following a procedure identified in the *Sweet Home Buildable Lands Inventory (2007).*[2] Projected water demands were proportionally distributed among the buildable vacant parcels and future developments based on the parcel's and/or project's area.

[2] Community Planning Workshop. April 2007. *Sweet Home Buildable Lands Inventory.*



Development Area	Percent of Total Demand	2043 ADD, gpm
A	0.2%	0.3
B	5.5%	9.5
C	0.2%	0.3
D	1.0%	1.7
E	0.8%	1.4
F	15.0%	26.0
G	8.0%	13.9
H	2.0%	3.5
I	4.0%	6.9
J	20.0%	34.7
K	2.0%	3.5
L	3.0%	5.2
M	0.3%	0.5
N	3.0%	5.2
O	0.5%	0.9
P	0.5%	0.9
Q	0.5%	0.9
R	2.5%	4.3
Vacant - Buildable	31.0%	53.8
Total	**100.0%**	**173.6**

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Development Areas

City Limit

Projected 2043 Average Day Demand

0 to 1 gpm (0 to 60 CCF/month)

1 to 5 gpm (60 to 300 CCF/month)

5 to 10 gpm (300 to 590 CCF/month)

10 to 15 gpm (590 to 880 CCF/month)

Greater than 15 gpm (880 CCF/month)

Notes:

1. Development Areas A through R are groupings of parcels which have been identified for near-term planned developments, as identified by City staff. All remaining growth areas are vacant parcels identified as "Buildable" following a process outlined in the Sweet Home Buildable Lands Inventory (2007).
2. The total projected increase in water use equal to 0.25 mgd (173.6 gpm) was allocated to parcels based on City input and the proportion of the total growth area.

N

0 900 1,800

Scale in Feet

WEST YOST

DRAFT

Figure 3-2

Future Development and Buildable Lands

City of Sweet Home

Water Master Plan

CHAPTER 4
Design and Performance Criteria

This chapter defines the recommended design and planning criteria to be used for evaluating the performance of the City's water distribution system and planning for future growth.

Key water system planning criteria have been incorporated into this chapter from the Oregon Drinking Water Services (DWS), Oregon Health Authority (OHA), the Environmental Protection Agency (EPA), the AWWA, and the Oregon Fire Code (OFC). The following sections of this chapter present the recommended planning criteria for the City's water distribution system:

- General Water System Recommendations
- Water System Capacity and Performance
- Facilities Sizing

Table 4-1 summarizes the recommended water system planning criteria for this WMP, which are discussed in more detail in the section below.

4.1 GENERAL WATER SYSTEM RECOMMENDATIONS

The City is concerned with providing reliable water service that meets all state and federal water quality requirements. Water quality standards and reliability are each discussed in the following sections.

4.1.1 Water Quality Standards

Water quality standards largely pertain to protecting public health and consistently delivering a satisfactory product to the customer. Most water quality considerations are related to supply and treatment issues and are not the subject of this plan. The EPA and Oregon DWS are responsible for establishing water quality standards and prescribe regulations that limit the amount of certain contaminants in water provided by a public water system. The City, as a water purveyor, is responsible for ensuring that the applicable water quality standards and regulations are always met. Requirements for routine system sampling of chlorine residual and prescribed contaminants may be found in the Oregon Administrative Rules (OARs) Chapter 333, Division 061. Additional water distribution system federal and state monitoring requirements are described below.

4.1.1.1 Distribution System Standards

The City complies with distribution system water quality monitoring and standards as prescribed by the EPA and Oregon DWS. In the water distribution system network, the Oregon DWS requires that there is a measurable chlorine residual level throughout the system in at least 95 percent of all monthly samples and a chlorine residual of at least 0.2 mg/L where water enters the distribution system. Additional routine sampling must be taken to verify maximum contaminant level (MCL) compliance for lead, copper, coliform, and disinfection byproducts. Routine distribution system sampling and requirements are further described below.

Table 4-1. City of Sweet Home Water System Planning and Design Criteria

Component	Criteria	Remarks / Issues
Fire Flow Requirement		
Residential		
Low Density Residential	1500 gpm @ 2 hours	–
Medium Density Residential	2000 gpm @ 2 hours	–
High Density Residential	3000 gpm @ 3 hours	–
Mixed Use		
Mixed Use Residential	3000 gpm @ 3 hours	–
Commercial		
Highway Commercial	3000 gpm @ 3 hours	–
Central Commercial	3000 gpm @ 3 hours	–
Planned Recreation Commercial	1500 gpm @ 2 hours	–
Industrial		
General Industry	3000 gpm @ 3 hours	–
Light Industrial	3000 gpm @ 3 hours	–
Heavy Industrial	4000 gpm @ 4 hours	–
Public		
Foster Elementary School	4500 gpm @ 4 hours	–
Hawthorne Elementary School	4000 gpm @ 4 hours	–
Oak Heights Elementary School	4000 gpm @ 4 hours	–
Junior High School	5500 gpm @ 4 hours	–
Sweet Home High School	5500 gpm @ 4 hours	–
Public - Open Space	1500 gpm @ 2 hours	–
Water Supply Capacity		
Supply/Pumping Capacity	Provide capacity equal to maximum day demand	–
Pumping Facility Capacity		
Pumping Capacity	Provide capacity equal to maximum day demand for the pressure zone	Design for peak hour or maximum day demand plus fire flow (whichever is larger), only if no gravity storage is available within the pressure zone.
Water Storage Capacity		
Operational Storage	25 percent of maximum day demand	–
Fire Storage	Varies dependent upon fire flow and duration of single largest possible fire event in pressure zone	1,500 gpm @ 2 hour = 0.18 MG
		5,500 gpm @ 4 hours = 1.32 MG
Emergency Storage	200 percent of maximum day demand	–
Total Water Storage Capacity	Operational + Fire + Emergency	–
Pipeline Sizing		
Diameter - Transmission	12-inches or larger	–
Diameter - Distribution	Less than 12-inches	–
Minimum Diameter	8-inches; 6-inches (dead-ends)	–
Maximum Pressure (psi)	120	According to the Uniform Plumbing Code, residences with pressures above 80 psi must have pressure reducing valves.
Minimum Pressure (psi)		
Average Day Demand	45	–
Maximum Day Demand	45	–
Maximum Day Demand plus Fire Flow	20	–
Peak Hour Demand	40	–
Maximum Pipeline Velocity (fps)		
Average Day Demand	5	New pipelines only.
Maximum Day Demand	5	New pipelines only.
Maximum Day Demand plus Fire Flow	12	New pipelines only.
Peak Hour Demand	5	New pipelines only.
Pipeline Material	PVC; DIP	–
Hazen Williams "C" Factor	130 (PVC); 120 (DIP)	For consistency in hydraulic modeling.

4.1.1.1.1 Final Lead Free Rule

Lead most commonly enters drinking water via service lateral pipelines, pipe fittings, and household plumbing fittings and fixtures. Excess lead in drinking water poses a public health risk, especially to vulnerable groups such as young children.

The United States Congress amended the Safe Drinking Water Act (SDWA) in 1986 to prohibit the use of pipes, solder, or flux that were not "lead free" in public water systems or any plumbing system that provides water for human consumption. Under the 2011 Reduction of Lead in Drinking Water Act (RLDWA), "lead free" was defined as a weighted average of the lead content of the wetted surfaces of plumbing products (e.g., pipes, pipe fittings, fixtures) less than 0.25 percent, and less than 0.2 percent lead for solder and flux; this decreased the allowable lead content allowed under the SDWA. The Final "Lead Free" Rule, published September 1, 2020 by the EPA, requires that manufacturers or importers certify that their products meet the definition of "lead free" using a consistent verification process within three years. The goal of this Rule is to reduce lead in drinking water and ensure that all parties, from regulators to consumers, have a common understanding of "lead free" plumbing. The City is required to use lead free products during the installation or repair of any public water system facility, as well as control the corrosivity of water through compliance with the Lead and Copper Rule.

4.1.1.1.2 Revised Total Coliform Rule

On April 1, 2016, the Oregon DWS began implementing provisions of the EPA Revised Total Coliform Rule (RTCR) with the intent of protecting the public from waterborne illness as a result of fecal contamination in distribution systems. The RTCR shifted MCL monitoring from total coliform to *E. coli*, as it is a more reliable indicator of fecal contamination. Under the RTCR, the *E. coli* MCL is considered exceeded if:

- The presence of *E. Coli* is confirmed (positive *E. coli* sample);
- Repeat samples are not tested after a positive *E. coli* or total coliform sample; or
- A total coliform-positive sample is not analyzed for *E. coli*.

Routine coliform monitoring is required monthly for public water systems that serve more than 1,000 people or use surface water as a supply source. If coliform bacteria are found during routine sampling, three additional repeat samples are required. These samples should be collected at the original tap with a coliform positive sample, and one tap each within five service connections upstream and downstream of the original tap. Additional or alternative sampling can be proposed by water suppliers at locations that present a likely pathway for contamination and should be identified in a Coliform Sampling Plan.

The RTCR also changed how coliform contamination is investigated and reported by water suppliers. The presence of total coliforms in a distribution system trigger Level 1 and Level 2 coliform investigations, rather than an immediate violation and notification to the public. Level 1 coliform investigations are triggered by:

- Two or more total coliform positive samples in the same month, if fewer than 40 samples are collected per month;
- The number of total coliform positive samples exceeds 5 percent if 40 or more samples are collected each month;
- Failure to collect the required repeat sample(s) after a single total coliform positive sample;

Level 1 coliform investigations consist of a self-assessment of the source water, treatment and distribution system, and operational practices, to determine potential sources of contamination. Level 2 coliform investigations are more detailed investigations performed by the applicable regulatory agency, and are triggered by:

- An *E. coli* MCL violation; or
- A second Level 1 coliform investigation within a rolling 12-month period
  - The regulatory agency may waive this criterion if a likely cause of the initial Level 1 investigation was identified by the regulatory agency, and corrected by the water supplier.

Operators must conduct a Level 1 investigation, or make themselves available for a Level 2 investigation, as soon as practical, correct any defects found, and submit the required forms to the Oregon DWS within 30 days after triggering a coliform investigation to avoid a violation and notice to water users.

It is expected that some samples will not be conclusively traced to a source of the contamination through investigations. This does not trigger a violation, but water suppliers are encouraged to perform actions such as flushing or additional sampling to help mitigate the issue. Regulators may require additional action if one or more coliform investigations are triggered within a rolling 12-month period, or four or more are triggered within a 24-month rolling period.

4.1.1.1.3 Stage 2 Disinfection Byproducts Rule

The Stage 2 Disinfection Byproducts Rule (DBPR) was introduced to reduce disease incidence associated with the disinfection byproducts (DBPs) that form when public water systems add disinfectants to potable water. This supplements the Stage 1 DBPR which established MCLs of 80 microgram per liter (µg/L) for trihalomethanes (TTHM) and 60 µg/L for the five major haloacetic acids (HAA5) based on a system-wide running annual average. The Stage 2 DBPR now bases compliance on the locational running annual average (LCAA) methodology, in which each monitoring station must not exceed the MCL, with the goal of reducing DBP exposure on a more equitable basis. Suppliers must conduct an initial distribution system evaluation (IDSE) to identify sites with high DBP level, which will become monitoring stations for Stage 2 DBPR compliance. The total number of LCAA monitoring sites is determine by the population served and should be geographically well distributed throughout the water system.

The City began Stage 2 monitoring in December 2013 at two monitoring stations. At the time of the preparation of this WMP, the City only monitors for DBPR compliance at one location.

4.1.1.2 Water Supply and Treatment Standards

The City complies with water quality monitoring and standards during treatment processes as prescribed by the EPA and Oregon DWS. Routine sampling must be taken at various points before and during the treatment processes to verify MCL compliance for turbidity, total organic carbon (TOC), pH, temperature, nitrate, arsenic, inorganic carbon (IOC), volatile organic compounds, synthetic organic chemicals, radionuclides, and nitrite. The City's water supply and treatment processes routinely meet the MCLs set for each chemical. Cyanotoxin monitoring is described in the following section to demonstrate the City's proactive approach to meeting water quality requirements. Specific sampling and reporting requirements can be found in OAR Chapter 333 Division 061, with additional guidance on resources provided on the Oregon Drinking Water Services website[1].

4.1.1.2.1 Cyanotoxins

Cyanotoxins encompass a range of toxins produced by cyanobacteria. Cyanobacteria are photosynthetic bacteria that "bloom" in surface waters, typically during summer and fall months, and can cause events commonly referred to as harmful algal blooms (HABs). Water suppliers are subject to OAR 333-061-0510 to 333-061-0580 if the source water is susceptible to HABs, and thus the release of cyanotoxins, and must monitor raw water intakes for cyanotoxins at least once every two weeks from May 1 through October 31. The health advisory levels of cyanotoxins are:

- Total Microcystins: 0.3 µg/L for vulnerable people; 1.6 µg/L for people aged 6 and older
- Cylindrospermopsin: 0.7 µg/L for vulnerable people; 3 µg/L for people aged 6 and older

For cyanotoxin levels greater than 0.3 µg/L, weekly raw water and finished water testing must occur weekly. If any finished water contains cyanotoxins, finished water testing must occur daily until two consecutive weeks of raw water samples measure below health advisory levels and no finished water contains detectable cyanotoxins. All cyanotoxin samples must be analyzed using the enzyme-linked immunosorbent assay (ELISA) for the specific cyanotoxin, EPA method 546, or another method approved in writing by the OHA. The OHA may revise (increase, decrease, or discontinue) the required cyanotoxin monitoring frequency at its discretion. OAR 333-061-070 specifies public notification requirements if cyanotoxin levels exceed health advisory limits in finished water.

On June 26, 2018, the State of Oregon issued a temporary administrative order in response to cyanotoxins found in the City of Salem's drinking water as a result of HABs in Detroit Lake. The City does not draw water from Detroit Lake but proactively sampled its finished water on June 15, 2018, and began sampling raw water bi-weekly on June 25, 2018. During this period, no cyanotoxins were detected in the City's raw water supply. The City is not required by OHA to monitor for cyanotoxins.

4.1.2 Water System Reliability

Water system reliability is achieved through a number of system features. Reliable systems include: appropriately-sized storage facilities; redundant or "firm" pumping and transmission facilities, where required; and alternate power supplies. Reliability and water quality are also improved by designing looped water distribution pipelines and avoiding dead-end distribution mains wherever possible. Looping pipeline configurations reduces the potential for stagnant water and the associated problems of poor taste and low disinfectant residuals. Proper valve placement is also necessary to maintain reliable and flexible system operation under normal and abnormal operating conditions.

[1] Accessed at https://www.oregon.gov/oha/PH/HEALTHYENVIRONMENTS/DRINKINGWATER/MONITORING/Pages/monitoring.aspx

4.2 WATER SYSTEM CAPACITY AND PERFORMANCE

Peak hour demand and maximum day demand plus fire flow conditions are used to assess the adequacy of the City's water system facilities and pipelines during high demand periods. Adopted peaking factors to represent maximum day and peak hour demands are discussed in *Chapter 2 Existing System Description*. The following subsections discuss the assumptions and criteria recommended to serve high demand conditions.

4.2.1 Fire Flow Requirements

Fire flow requirements were developed with input from the City to be generally consistent with the 2019 Oregon Fire Code, Tables B105.1 and B105.2, which establish minimum fire flows and durations for individual structures based on the structure's construction type and fire flow calculation area. The fire flow requirements presented in this WMP have not been developed for specific structures and are intended only for general planning purposes. All recommended fire flows were approved by the Sweet Home Fire District and City staff.

Table 4-2 summarizes the recommended minimum fire flow requirements by Comprehensive Plan land use. Fire flows shall be met concurrently with a maximum day demand condition, while maintaining a minimum distribution system residual pressure of 20 pounds per square inch (psi). Fire flows and the expected duration will also be used to establish treated water storage requirements.

It should be noted that land uses designated as "Public" range widely in both the type and density of structures. Therefore, the minimum required fire flow was increased for schools, as listed in Table 4-2, based on input from City staff familiar with each location's structure type and size.

4.2.2 Water Supply and Treatment Capacity

Appropriate criteria to assess the adequacy of the water supply during high demand periods are:

- Maximum Day Demand: The water supply system (raw water intake, water treatment, and finished water pumping) should be able to produce a maximum day demand.
- Peak Hour Demand: The water distribution system (a combination of treated surface water from the water treatment plant and water from the distribution storage tanks) should be able to deliver a peak hour demand.

Comprehensive Plan Land Use[a]	Fire Flow, gpm	Duration, hours	Recommended Storage, MG
Residential			
Low Density Residential	1,500	2	0.18
Medium Density Residential	2,000	2	0.24
High Density Residential	3,000	3	0.54
Mixed Use			
Mixed Use Residential	3,000	3	0.54
Commercial			
Highway Commercial	3,000	3	0.54
Central Commercial	3,000	3	0.54
Planned Recreation Commercial	1,500	2	0.18
Industrial			
General Industry	3,000	3	0.54
Light Industrial	3,000	3	0.54
Heavy Industrial	4,000	4	0.96
Public[b]			
Foster Elementary School	4,500	4	1.08
Hawthorne Elementary School	4,000	4	0.96
Oak Heights Elementary School	4,000	4	0.96
Junior High School	5,500	4	1.32
Sweet Home High School	5,500	4	1.32
Public - Open Space	1,500	2	0.18

Table 4-2. Fire Flow Requirements

(a) Land use designations are based on the City of Sweet Home Comprehensive Plan, amended 8/27/2010.
(b) A more stringent fire flow requirement is assigned to schools due to the size of the structures in relation to surrounding land uses.
MG = Million Gallons

4.2.3 Distribution System Pressures

Adequate system pressure is a basic indicator of acceptable water distribution system performance. The recommended planning criteria for system pressures are:

- Allowable Pressures Under Normal Operating Conditions: 40 psi to 120 psi[2]
  - Minimum Pressure under Average Day Demand: 45 psi
  - Minimum Pressure under Maximum Day Demand: 45 psi
  - Minimum Pressure under Peak Hour Demand: 40 psi
- Minimum Pressure Under Fire Flow Conditions: 20 psi

These performance criteria are applied to all areas that fall within the normal customer service elevation ranges for each pressure zone. Customers above or below the normal service elevation ranges may require an individual pressure reducing valve or booster pump.

4.3 FACILITIES SIZING

The following sections describe the recommended criteria governing the size of water facilities (i.e., pump stations, storage reservoirs, and pipelines) within the City's service area.

4.3.1 Pumping Facility Capacity

Sufficient water system pumping capacity should be provided to meet the demands of the pressure zone. For zones with storage, sufficient pumping capacity should be provided to meet the maximum day demand for the pressure zone. For pressure zones without storage, sufficient pumping capacity should be provided to meet the greater of the following demand conditions within the zone:

- A peak hour demand; or
- A maximum fire flow event concurrent with the maximum day demand.

The analysis of pumping facility capacity should be conducted assuming the largest booster pump is out of service (i.e., firm capacity of the pump station). This assumption ensures reliable deliveries during high demand conditions. Pump stations with only one booster pump will not be considered reliable in a high demand condition.

Critical pumping facilities are defined as those facilities that provide service to pressure zone(s) and/or service area(s) which do not have sufficient fire and/or emergency storage available and meet the following criteria:

- The largest pumping facility that provides water to a particular pressure zone and/or service area; or
- A facility that provides the sole source of water to a single pressure zone and/or service area.

[2] The Uniform Plumbing Code (UPC) requires that individual services that exceed 80 psi have an individual pressure regulator on the service line; services that are less than 40 psi during an average day demand condition must have an individual booster pump on the service line.

All critical pumping facilities should be equipped with an on-site backup power generator.

4.3.2 Water Storage Capacity

Total treated water storage capacity requirements are evaluated based on the following three components:

- Operational Storage
- Fire Storage
- Emergency Storage

Each storage component is discussed below.

4.3.2.1 Operational Storage

Over any 24-hour period, water demands will vary. Typically, higher water demands will occur during the early morning hours when users are irrigating landscape and getting ready to go to work and school. Water demands will then decline to some nominal baseline level (depending on the proximity to and water use patterns of adjacent commercial/industrial areas) before increasing depending on outside water needs (and corresponding temperature) and again reaching a higher water demand in the early evening hours as people return home. Throughout the year, the peaks of this cycle will vary according to customer needs, with the largest peak occurring in the summer, creating the maximum day and peak hour demands for which the system should be designed.

The City operates its WTP intermittently over a 24-hour period. Additional flow is provided from storage tanks during these periods when the WTP is offline, as well as during peak demand periods when the WTP is operating. Storage tanks are typically replenished when demands drop below the WTP water supply. The storage volume used to meet the difference between demand and supply during the peak demand periods or when the WTP is off is called operational storage.

For a typical system, the volume of water recommended to be held in reserve for operational flow should be at least equal to 25 percent of the total volume of water used on the maximum day.[3]

4.3.2.2 Fire Storage

Fire storage is the volume of storage reserved for fire flows. The fire storage volume is determined by multiplying the required maximum fire flow rate by the required duration. It is assumed that no more than one fire flow event would occur in any pressure zone at one time.

4.3.2.3 Emergency Storage

A storage reserve is required to meet demands during an emergency. 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. Determination of the required volume of emergency storage is a policy discussion based on the assessment of the risk of failures and the desired degree of system reliability. The amount of required

[3] AWWA Manual M32, Distribution Network Analysis for Water Utilities *(AWWA, 2012) states that for large systems, the equalizing storage requirement is typically 15 to 20 percent of the total maximum day demand over a 24-hour period, but equalizing storage could exceed 30 percent for small service areas or arid climates (page 116).*

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.

The AWWA states that no formula exists for determining the amount of emergency storage required, and that the decision will be made by the individual utility based on a judgment about the perceived vulnerability of the system. The City has recently experienced minor emergencies (e.g., main breaks to isolated areas, power failure, etc.), in which existing storage was the sole supply source. The City does not have adequate storage/redundancy for multiple days of service. Furthermore, the City's power utility may institute rolling blackouts during severe wildfire conditions, typically in the summer and fall, which could last for several days. For this WMP, it is recommended that the City have a minimum quantity of emergency storage volume equivalent to 200 percent of the maximum day demand.

4.3.2.4 Total Storage Capacity Recommended

The City's recommended total water storage capacity is the sum of the following components:

- **Operational**: Volume of water necessary to meet diurnal peaks observed throughout the day, assumed to be equivalent to at least 25 percent of the maximum day demand; plus
- **Fire Flow**: Volume of water necessary to supply a fire flow event, where the fire flow event is contingent upon the land use designation; plus
- **Emergency**: Volume of water necessary to provide an emergency supply of 200 percent of the maximum day demand.

The amount of total system storage required to meet these criteria will change over time as water demands within the City change.

4.3.3 Pipeline Sizing

The following criteria will be used as guidelines for sizing transmission and distribution system pipelines. Although these criteria and guidelines have been established and will be used to size new pipelines, the City's existing water system should be evaluated using system pressure as the primary criterion. Secondary criteria, such as pipeline velocity, head loss, age, and material type, are used as indicators to locate, and to help prioritize where water system improvements may be needed.

Water pipelines should be sized based on the criteria described below for average day, maximum day plus fire flow, and peak hour demand conditions. Existing pipelines are assumed to have been designed to meet earlier standards in place at the time of installation.

4.3.3.1 General Definitions and Standards

The following list summarizes the general definitions and City standards for pipelines:

- Transmission pipelines are generally greater than or equal to 12-inches in diameter.
- Distribution pipelines are generally less than 12-inches in diameter.
- All new pipelines are required to be PVC or ductile iron pipe (DIP).

- All new pipelines are required to have a minimum diameter of 8-inches, or 6-inches for dead-end mains only.[4]

4.3.3.2 Average Day Demand

West Yost recommends evaluating average day demand conditions using the following planning criteria:

- Pressures should be maintained between 45 and 120 psi at the customer service elevation. According to the Uniform Plumbing Code, residences with pressures above 80 psi must have pressure reducing valves.
- The maximum velocity within new pipelines should be 5 feet per second (fps).

4.3.3.3 Maximum Day Demand

West Yost recommends evaluating maximum day demand conditions using the recommendations listed in *Section 4.3.3.2.*

4.3.3.4 Maximum Day Demand plus Fire Flow

West Yost recommends evaluating maximum day demand plus fire flow conditions using the following planning criteria:

- The minimum allowable service pressure in the water distribution system should be maintained at 20 psi.
- The maximum velocity within new pipelines should be 12 fps.

4.3.3.5 Peak Hour Demand

West Yost recommends evaluating peak hour demand conditions using the following planning criteria:

- The minimum residual pressure during a peak hour demand should be 40 psi.
- The maximum velocity within new pipelines should be 5 fps.

[4] The City does permit pipelines as small as 3 inches on a case-by-case basis; this only applies if the pipeline serves low demands where a 6-inch pipeline would cause low chlorine residuals or other water quality issues.

CHAPTER 5
Hydraulic Model Update

This chapter describes the hydraulic model update and the subsequent steady-state calibration process performed to confirm that the updated model can accurately represent the City's existing water system under varying conditions. The resulting updated hydraulic model was used to evaluate the adequacy of the City's water system under future water demand conditions in *Chapter 6 Water System Analysis.*

The hydraulic model updates, calibration, and verification efforts are described below in the following sections:

- Hydraulic Model Background
- Hydraulic Model Update Methodology
- Review and Update of the Hydraulic Model
- Hydraulic Model Calibration
- Summary of Findings and Conclusions

5.1 HYDRAULIC MODEL BACKGROUND

The City's hydraulic model was developed by Murraysmith in 2020[1] using the Innovyze InfoWater Pro® software. West Yost converted the InfoWater Pro® model to InfoWater® in 2021 for use in developing the City's Small Diameter Water Main Replacement Program (SDM Program). The model is a reduced all-pipe model, whereby all distribution pipes are included based on the City's water pipes GIS shapefile, but individual hydrants are not represented as individual nodes and service lateral pipelines are generally not included.

As part of the development of this WMP, a comprehensive hydraulic model update was performed to create the most current representation of the City's existing water system. Information for pipelines and major facilities (such as valves, pumps, and tanks) was updated with the most current records provided by the City. Updated water demands calculated in *Chapter 3 Water Demand* were allocated to junctions in the hydraulic model using spatially-located water meter billing data, and the hydraulic model was calibrated to ensure its ability to represent the City's water system. Each component of the hydraulic model update process is described in the sections below.

5.2 HYDRAULIC MODEL UPDATE METHODOLOGY

To update the existing water system hydraulic model, West Yost performed the following key tasks:

- Updated existing pipelines and added new pipelines;
- Reviewed and updated system connectivity with City input;
- Updated existing water system facilities (e.g., storage reservoirs and pump stations);
- Allocated existing water demands using the City's spatially-located meter and billing information;
- Developed a hydrant testing plan to collect hydrant flow and pressure data, which was executed by City Operations staff on January 19 and 20, 2022; and
- Calibrated the hydraulic model with results from data collected during hydrant testing.

[1] *Sweet Home Water Distribution and Treatment Steady State Hydraulic Model Calibration,* Murraysmith, March 4, 2020.

To accomplish these tasks, West Yost worked closely with the City's Public Works Department to obtain and review the following:

- Information on existing storage tanks, pumping facilities, water supply, and water treatment facilities;
- Drawings associated with recent water system improvements;
- "Near-term" capital improvement projects expected to be constructed during or shortly after completion of the WMP, and considered "existing" for purposes of this WMP;
- The City's GIS database of water system facilities (e.g., pipelines, hydrants, valves, etc.), provided November 24, 2021;
- Current water system operations (e.g., WTP operating patterns, inactive facilities, etc.), as provided by the City via telephone interviews and email communications;
- Metered account and billing information; and
- Historical Supervisory Control and Data Acquisition (SCADA) system screenshots.

5.3 REVIEW AND UPDATE OF THE HYDRAULIC MODEL

The following sections describe the findings of West Yost's model review and highlight the specific updates that were performed to best replicate existing system conditions.

5.3.1 InfoWater® Conversion

The City's existing InfoWater Pro® hydraulic model was directly converted to InfoWater® using the InfoWater Database folder as the basis for the conversion to preserve all model data.

5.3.2 Pipeline Roughness Factors

Typically, pipeline roughness factors, or C-factors, are assigned based on the characteristics of a pipeline, such as material, diameter, and/or installation date (age). The City's existing hydraulic model contained C-factors significantly higher (i.e., less rough) than industry-accepted C-factors for similar pipelines and therefore may not have been representative of true field conditions. Industry-accepted C-factors generally align with calibrated roughness factors maintained in West Yost's database of C-factors, which has been developed to summarize C-factors from previous hydrant tests for different material types, diameters, and ages. As part of the SDM Program, West Yost initially updated C-factors in the City's model per the C-factor database. Table 5-1 presents the preliminary C-factors assigned to each of the different pipeline material types within the City's water system. These C-factors were then confirmed or adjusted during the calibration of the hydraulic model, which is discussed further in *Section 5.4*.

Table 5-1. Preliminary Pipeline Roughness C-Factors Assigned in Hydraulic Model			
Pipeline Material Type	Acronym	Hazen-Williams C-factor	
		Diameter < 12-inches	Diameter ≥ 12-inches
Cast Iron	CI	75[(a)]	100
Ductile Iron	DI	130	140
Galvanized Steel	GALV	120	-
Polyvinyl Chloride	PVC	140	
Steel	STL	120	
Unknown	UNK	120	
(a) The C-factor for Cast Iron pipelines less than 12-inches was increased to 90 based on hydrant test results, as discussed in Section 5.4.2.			

5.3.3 Existing System Facilities and Pipelines

Based on a review of the available facilities and pipeline data for the existing and near-term water system, the following facilities were added or updated in the City's current hydraulic model:

- Updated pipeline connectivity and configuration issues identified with InfoWater® Connectivity and Network Review/Fix tools (based on City staff input).
- Added or abandoned hydraulic model pipelines to remain consistent with the City's most recent GIS geodatabase, which had been updated since the hydraulic model was built in 2020.
- Updated pipelines with incorrect diameters, installation/retirement years, and/or C-factors based on City's most recent GIS data, as-built drawings, near-term improvements, and City staff input.
- Updated reservoir diameters and minimum and maximum elevations based on as-built drawings.
- Updated pump curves based on as-built drawings and manufacturer information.
- Updated junction elevation using a light detection and ranging (Lidar) digital elevation model (DEM) provided by the City on November 9, 2021.
- Updated pump elevations based on as-built drawings.

5.3.4 Spatially Located Meter Accounts

City staff provided West Yost with a billing database file containing a list of metered accounts and the corresponding metered water consumption data by account number, billing period, meter read, customer billing class, service code, and service address for each month from 2016 through 2020. A separate water meter GIS file was provided by City staff to link the metered water consumption data to spatially-located water meters. Based on discussions with City staff, it was decided to use the metered water consumption data from calendar year 2020 to allocate existing water demands to the hydraulic model to capture the most current spatial distribution of water demands.

Over 97 percent of the 2020 water consumption data was assigned a spatial location after linking the billing data to the City's spatially-located water meters. Figure 5-1 shows the spatial distribution of the meter demand data that was used to update the model. Approximately three percent of the 2020 water use remains unlocated. The spatially-located demands were scaled up (globally adjusted) to match the total water produced by the City in 2020 (0.85 mgd) to account for the unlocated meters and non-revenue water (see *Chapter 3 Water Demand*).

5.3.5 Water Demand Allocation

Average day water demands for calendar year 2020 were allocated in the hydraulic model by pressure zone using the spatially-located meter account data. InfoWater®'s Demand Allocator Tool analyzes the metered demand data to identify the closest pipeline to each meter point. The tool then applies the metered water demand to the closest junction of the selected pipeline. West Yost staff reviewed the allocated water demands to confirm that the demands were allocated properly by pressure zone. Demands for large water users (i.e., the City's WWTP) were also confirmed to be allocated to the correct pipeline.

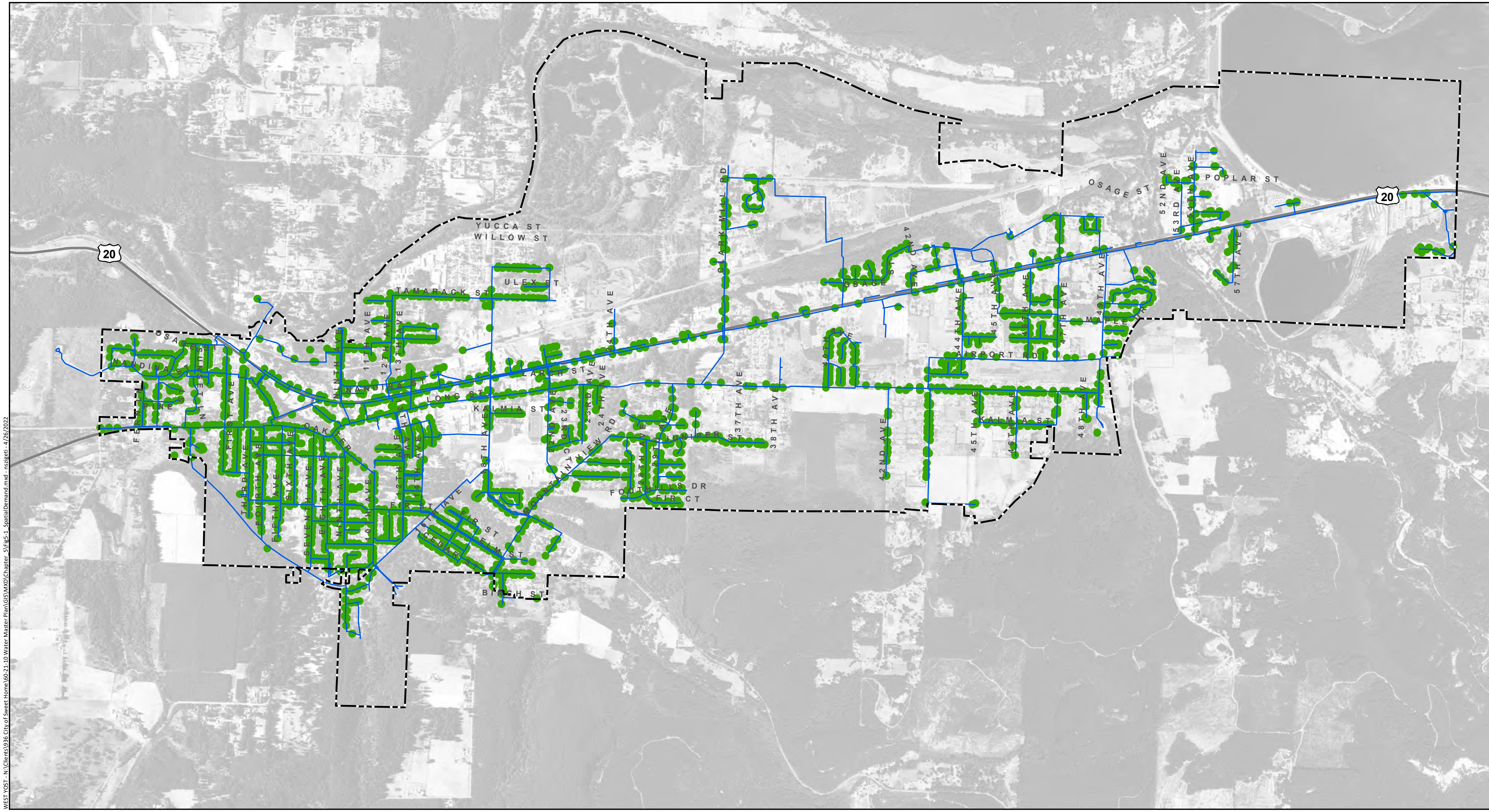
5.4 HYDRAULIC MODEL CALIBRATION

Steady-state calibration of the hydraulic model used data gathered through hydrant tests to confirm if: 1) the preliminary pipeline roughness factors (C-factors) that have been assigned to pipelines in the City's hydraulic model are appropriate; and 2) the City's hydraulic model can accurately simulate fire flow conditions. Depending on the model simulation results, pipeline C-factors may be adjusted in the hydraulic model to better reflect observed field conditions. West Yost prepared a memorandum summarizing the recommended hydrant test locations and procedures on December 9, 2021, which is included in Appendix A. The following sections discuss the hydrant testing program and the hydraulic model calibration results.

5.4.1 Hydrant Testing Program

Eighteen (18) locations were chosen for hydrant flow testing. Table 5-2 lists the locations of each test and their field status. The selection of these hydrant tests was based on pipeline diameter, proximity to pressure zone boundaries and water system facilities, surrounding pipeline characteristics (i.e., diameter, material, age), and regions with high elevations or remote (hydraulically distant from supply) locations. The final test locations are shown on Figure 5-2.

Hydrant flow testing was performed on January 19 and 20, 2022, by City Operations staff. All but two of the 18 scheduled tests were successfully performed. One test (Hydrant Test #8) was cancelled for unknown reasons and the static pressures were not recorded. Another test (Hydrant Test #18) was performed but the hydrant discharge flow was not recorded. The missing data from Hydrant Test #18 is considered insignificant since this test evaluates the LakePointe Pressure Zone, a very small zone (i.e., fewer than 20 customers) served by pipelines constructed in 2008 and a pump station and hydropneumatic tank constructed in 2002. Due to the age of its facilities and number of customers served, the LakePointe Pressure Zone will not be evaluated as part of the system analysis in this WMP.



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- Spatially Located Demand
- Water Pipelines
- City Limits

DRAFT

0 900 1,800
Scale in Feet

WEST YOST

Figure 5-1

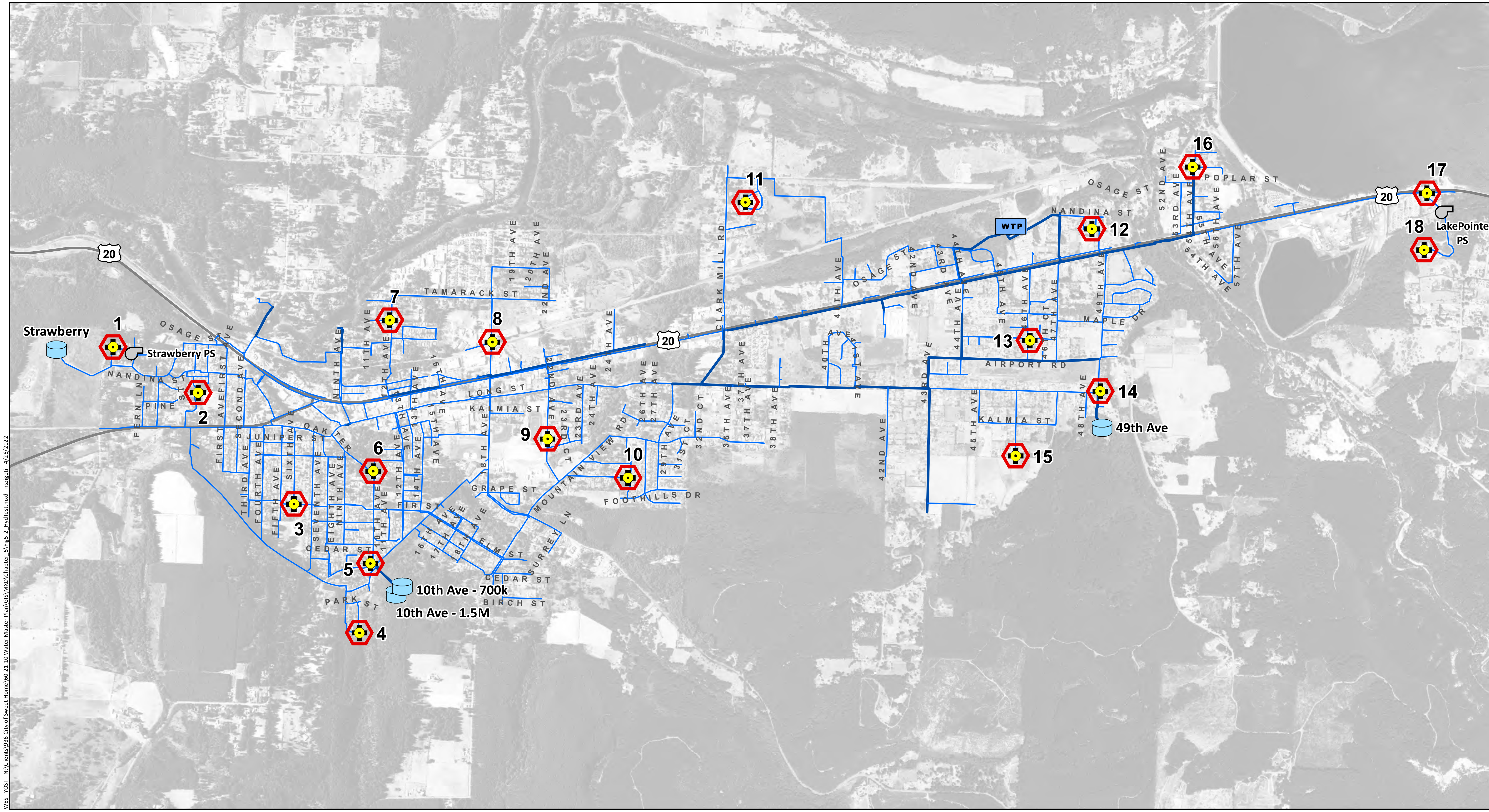
Spatially Located Water Demands

City of Sweet Home
Water Master Plan

Table 5-2. Hydrant Test Locations

Hydrant Test No.	Approximate Location	Comments	Field Status
1	1459 Strawberry Ridge	Strawberry Pressure Zone	Completed
2	1321 Sunset Lane	High elevation	Completed
3	610 Elm Street (across from Oak Heights Elementary)	High elevation	Completed
4	Corner at Taylor Creek Drive and Timber Street	High elevation; dead end	Completed
5	960 Alder Street (intersection of 10th Avenue and Alder Street)	Downstream of 10th Avenue tanks	Completed
6	745 10th Avenue	1950's 10-inch cast iron	Completed
7	1806 12th Avenue	Isolated area	Completed
8	1621 18th Avenue (near railroad tracks)	1940's 6-inch cast iron	Cancelled
9	951 22nd Avenue	1960's-1970's 6-inch ductile iron	Completed
10	778 27th Avenue	1970's-2000's 6-inch to 8-inch Ductile iron	Completed
11	1941 37th Circle	1980's-2000's 8-inch ductile iron	Completed
12	4879 48th Loop	Near water treatment plant	Completed
13	1219 46th Avenue	8-inch PVC	Completed
14	1199 49th Avenue	Downstream of 49th Avenue tank	Completed
15	1083 46th Avenue (at bend in 46th Avenue)	1980's 6-inch to 8-inch ductile iron	Completed
16	1702 54th Avenue	Isolated area	Completed
17	Intersection of Highway 20 and Riggs Hill Road	At end of long dead-end main	Completed
18	6309 LakePointe Way (in cul-de-sac)	LakePointe Pressure Zone	Flow not recorded[(a)]

(a) Static pressures were obtained for Hydrant Test #18.



Test Hydarnt

Water Treatment Plant

Storage Tank

Pump Station

Water Pipelines

Less than 12-inch

12-inch and Greater

DRAFT

Scale in Feet: 0 — 900 — 1,800

WEST YOST

Figure 5-2

Hydrant Test Locations

City of Sweet Home

Water Master Plan

WEST YOST - N:\Clients\936 City of Sweet Home\60-21-10 Water Master Plan\GIS\MXD\Chapter_5\Fig5-2_HydTest.mxd - nszigeti - 4/26/2022

Each hydrant test consisted of flowing water from an identified test hydrant to observe how the City's water system responds to fire flow conditions. The testing procedure consisted of monitoring the discharge flow and pressure at the key (flowing) hydrant and the pressures at other observed hydrants along the supply route(s) to the key hydrant. Static pressures were measured while the key hydrant was closed, and residual pressures were measured while the key hydrant was flowing. No isolation valves were closed for these hydrant tests. Each test typically had two to three observation hydrants, denoted by the test number and then an alphabetical designation. For example, in Test 1, the key hydrant is "1", and the two observation hydrants are "1A" and "1B."

City staff provided SCADA system screenshots for the WTP finished water pumps, the LakePointe Pump Station, the Strawberry Reservoir, and the 49th Avenue Reservoir. SCADA for the Strawberry Pump Station and 10th Avenue Reservoirs was not available during the testing period. City staff also provided WTP daily production data for January 2022. This information on the operations of the City's water system during testing was used to determine the City's overall water demand during the testing period (approximately 0.95 mgd) and to set up the boundary conditions in the hydraulic model.

Each completed test was simulated using the hydraulic model of the City's water system. Model-simulated results were compared to the observed field data to determine the accuracy of the hydraulic model. The differences between the observed static and residual pressures for the field hydrant tests were calculated and compared to the pressures predicted by the model. The goal of the calibration effort was to achieve no more than 5 psi pressure differential between the field data and the model-simulated results, which is based on standard engineering practice for model calibration in water system planning. Results from the hydrant testing program are discussed below.

5.4.2 Hydraulic Model Calibration Results

The results of the simulated hydrant flow tests generally validate the water system pipeline configuration and indicated that an adjustment to the preliminary C-factor assigned to cast iron pipelines was required. The C-factor for cast iron pipelines less than 12-inches in diameter was increased from 75 to 90 (i.e., less rough) after the flowing residual results indicated that preliminary pipeline losses were too high. A summary of the hydraulic model calibration results is provided in Table 5-3.

Of the 16 tests that were conducted, seven of the hydrant tests required further review and evaluation because they did not initially meet the ±5 psi tolerance limit for calibration as discussed below. Two of the seven tests identified for further review (Hydrant Tests #10 and #11) were evaluated under assumed backwash and 49th Avenue Reservoir filling operations, as described in the sections below. These operations will likely change when: 1) the new WTP backwash pump improvements are constructed; and 2) improvements are implemented to better operate the 49th Avenue Reservoir, which currently fills too quickly and is manually throttled at the butterfly valve located halfway up the 16-inch the reservoir supply pipeline.

5.4.2.1 Hydrant Test #1

Static pressures for this hydrant test were well-calibrated, but the differences between field-observed and model-simulated differential pressures were above the ±5 psi tolerance limit for Hydrant 1B. Pressure losses observed in the field at Hydrant 1B were 8 psi larger than those simulated by the model. These results indicate that there could be a partially closed valve in the field along the pipeline between Hydrant 1A and the flowing hydrant.

The hydraulic model was updated with this assumption, and the revised results are within the ±5 psi tolerance limit as shown in Table 5-3. It is recommended that City staff confirm the status of the inline valve located at the corner of Strawberry Loop and Strawberry Ridge (i.e., the valve identified in the City GIS as Asset ID "Valve1005").

5.4.2.2 Hydrant Test #7

Model-simulated static pressures for this hydrant test were calibrated to within ±5 psi of the field-observed pressures, but the differences between field-observed and model-simulated differential pressures were above the ±5 psi tolerance limit for Hydrant 7B. Pressures observed in the field at Hydrant 7B were unexpectedly reported to increase by 3 psi while Hydrant 7 was flowing; however, the Hydrant 7B model-simulated residual pressures decreased by 17 psi from static pressures, resulting in a comparison of differential pressures with losses of 20 psi greater in the hydraulic model than in the field.

These results indicate a possible error (e.g., faulty pressure gauge) in field-observed residual pressure readings for Hydrant 7B. The residual pressure increased while the test hydrant was flowing during a period when losses would be anticipated in the system. Since Hydrant 7B is located at the end of a 6-inch pipeline downstream from the flowing hydrant, it should not exhibit a pressure increase based on local system hydraulics. In addition, the static hydraulic grade at Hydrant 7B is approximately 17 feet lower than the static hydraulic grade at Hydrants 7A and 7C. Since all observation hydrants are located in close proximity, the static pressures at Hydrants 7A, 7B, and 7C should be similar.

No adjustment in pipeline C-factors is recommended due to a suspected defective pressure gauge reading at Hydrant 7B. It is recommended that the City checks the accuracy of the pressure gauges used for hydrant testing to ensure that they are correctly calibrated for future use.

5.4.2.3 Hydrant Test #10

The City backwashes the WTP filters on distribution system pressure. During backwash operations, approximately 3,200 gpm flows into the WTP backwash supply pipeline, bypassing the finished water pumps and backwashing the filter units using distribution system pressure. These operations generally result in a reduced distribution system pressure for a short period of time (i.e., five minutes), which is relatively short in comparison to the overall hydrant test duration.

This test was initially modeled under full backwash conditions, assuming a 3,200 gpm demand at the WTP, consistent with notes provided by the City that indicated a backwash was in effect during the test. However, neither the static pressures nor the differences between field-observed and model-simulated differential pressures were within the ±5 psi tolerance limit for all hydrants in this test. These results indicated that the boundary conditions (i.e., backwashing from distribution system pressure) were inadequate to accurately model this scenario.

It is possible that the backwash operation occurred during this test over a short interval of time relative to the full duration of Hydrant Test #10. Therefore, the backwash demand would not have drawn from the distribution system for the full duration of the hydrant test. As indicated in the field notes, static pressures were recorded over a span of nearly ten minutes—during 2:26 PM, 2:30 PM, and 2:35 PM (which was indicated to be the backwashing timestep). The static pressures should be relatively constant for all hydrants, as they are at similar elevations, but the static pressures vary by up to 6 psi between Hydrant 10A and Hydrants 10B/10C, which might indicate that the system has not reached static equilibrium between backwash and normal operating conditions.

The results shown in Table 5-3 assume that the WTP finished water pumps are offline, no backwash is occurring, and the 49th Avenue Reservoir operates as described in Section 5.4.2.4. As shown, the field-observed differential pressure at Hydrant 10A is 12 psi, or 7 psi larger than the model-observed differential between the static and residual pressure with no backwash condition. However, a 12 psi differential between static and residual pressures is observed in the hydraulic model if a WTP backwash is assumed to occur. Due to the uncertainty between described and actual operations, no adjustment to C-factors is recommended.

5.4.2.4 Hydrant Test #11

The differences between field-observed and model-simulated differential and static pressures were initially above the ±5 psi tolerance limit for all hydrants. Upon further review of the hydraulic model, it was determined that operation of the 49th Avenue Reservoir must be modeled differently when draining versus filling. Adjustments to the simulated operations at the 49th Avenue Reservoir are described below.

Generally, the City actively manages the turnover of the Main Zone reservoirs (i.e., 49th Avenue and 10th Avenue Reservoirs) using the WTP finished water pumps. The WTP finished water pumps are controlled by the level of the 49th Avenue Reservoir. The 10th Avenue Reservoirs are sited at a hydraulically distant location from the WTP and fill more slowly than the 49th Avenue Reservoir despite being sited at the same elevation. If system operations are not evaluated and adjusted seasonally, the 49th Avenue Reservoir will generally overflow before the 10th Avenue Reservoirs can fill. To prevent the rapid rate of fill at (and subsequent overflow of) the 49th Avenue Reservoir, the City manually throttles a valve on the combined inlet/outlet 16-inch PVC pipeline that serves the reservoir. The valve position (i.e., degree throttled) is adjusted seasonally based on system demands. The hydraulic model was updated to replicate these operations by adding a throttled valve on the combined inlet/outlet pipe and iterating the degree throttled using field static pressures during filling operations as a target value. By applying large minor losses to the throttled valve at the 49th Avenue Reservoir, back-pressure is created in the east side of the City when the WTP finished water pumps are supplying the water system and filling the reservoirs. Static pressures in the hydraulic model for all tests under these conditions generally calibrate to within ±5 psi of the field-observed static pressures.

While the hydraulic model was able to replicate most tests under reservoir filling operations (i.e., a WTP finished water pump is operating), the assigned large minor losses did not allow the 49th Avenue Reservoir to drain quickly enough to sufficiently supply the flowing hydrant in the hydraulic model. Based on these findings, the minor losses assigned to the throttled valve for Hydrant Tests #10 through #12 were reduced to allow more supply from the 49th Avenue Reservoir into the system. The discrepancy between filling and draining operations could be caused by another throttled valve on the inlet pipe to the 49th Avenue Reservoir, in addition to the throttled valve on the combined inlet/outlet pipe. The hydraulic model was updated with the assumption that two valves are throttled—one on the combined reservoir inlet/outlet pipe (i.e., a reduced minor loss during draining) and one on the dedicated inlet pipeline (i.e., a larger minor loss during filling)—and the revised results are shown in Table 5-3. The revised 49th Avenue operations were validated by the results of Hydrant Test #12.

The revised model operations resulted in only one of the three observed hydrants remaining above the ±5 psi tolerance limit threshold for differences between field-observed and model-simulated differential pressures. However, the observed static pressure in the field at Hydrant 11B is 8 psi lower than the local static pressures at Hydrants 11 and 11A, which are sited at a similar elevation. Due to the varying observed static pressures between Hydrant 11B and Hydrants 11 and 11A, it is possible that the pressure gauge used on Hydrant 11B was faulty. Although the pressure discrepancies for this test cannot be fully explained at this time, it should be noted that Hydrant Test #12 is well-calibrated under the same 49th Avenue Reservoir draining operations. Therefore, no adjustment in pipeline C-factors is recommended. It is recommended that the City checks the accuracy of the pressure gauges used for hydrant testing to ensure that they are correctly calibrated for future use.

5.4.2.5 Hydrant Test #13

The results of Hydrant Test #13 are shown in Table 5-3. Static pressures for this hydrant test were well-calibrated, but the differences between field-observed and model-simulated differential pressures were above the ±5 psi tolerance limit for Hydrant 13B only. Pressure losses observed in the field at Hydrant 13B were 6 psi larger than those simulated by the model. The supply to the hydrant is provided by three 8-inch pipelines, on which all three observation hydrants are sited. Under flowing conditions, all three supply paths should exhibit similar headlosses (i.e., pressure drops), as shown in the model. However, losses exhibited in the field were 40 percent higher at Hydrant 13B.

These results indicate a possible error (e.g., faulty pressure gauge) in field-observed residual pressure readings for Hydrant 13B. Although unlikely, it is also possible that multiple partially closed valves exist in the vicinity of the test. Model-simulated differential pressures are within ±5 psi of the field-observed differential pressures if valves are closed: 1) in 46th Avenue, between the flowing hydrant and Hydrant 13A; and 2) in Live Oak Street, between Hydrant 13B and 47th Avenue.

No adjustment in pipeline C-factors is recommended since all pipelines in this area are PVC pipes constructed since 2000. It is recommended that the City checks the accuracy of the pressure gauges used for hydrant testing. If the discrepancies cannot be explained by faulty pressure gauges, it is recommended that City staff confirm the status of the valves located in 46th Avenue and Live Oak Street.

5.4.2.6 Hydrant Test #14

The results of Hydrant Test #14 are shown in Table 5-3. The differences between field-observed and model-simulated differential pressures were above the ±5 psi tolerance limit for Hydrant 14B only. Pressure losses observed in the field at Hydrant 14B were 6 psi larger than those simulated by the model. It is possible that there were errors in pressure readings at this test since the field-observed static hydraulic grade at Hydrants 14, 14A, and 14B varies by over 20 feet between Hydrant Tests 14 and 14A. Typically, the static hydraulic grade at nearby hydrants should be similar when served by pipes with few losses (i.e., large diameter pipelines under non-flowing conditions).

No adjustment in pipeline C-factors is recommended since all pipelines in this area are PVC or DI and the C-factors have been calibrated in other tests. It is recommended that the City checks the accuracy of the pressure gauges used for hydrant testing to ensure that they are correctly calibrated for future use.

5.4.2.7 Hydrant Test #16

The results of Hydrant Test #16 are shown in Table 5-3. Static pressures for this hydrant test were well-calibrated, but the differences between field-observed and model-simulated differential pressures were above the ±5 psi tolerance limit for Hydrant 16B only. Pressure losses observed in the field at Hydrant 16B were 6 psi larger than those simulated by the model. It is possible that there were errors in pressure readings at this hydrant since Hydrant 16B is sited on a looped pipeline that does not serve as a primary supply to the flowing hydrant and therefore should not experience high pressure losses in the field.

Table 5-3. Summary of Hydrant Test Calibration Results

Hydrant	Field Data			Modeled Data			Comparison of Differential Pressures (Field - Model)
	Static Pressure, psi	Residual Pressure, psi	Differential Pressure, psi (Static - Residual)	Static Pressure, psi	Residual Pressure, psi	Differential Pressure, psi (Static - Residual)	
Hydrant Test No.1							
Flowing 1	46	No Data	No Data	49	40	9	-
1A	56	50	6	52	49	3	3
1B	70	53	17	68	59	9	8
Hydrant Test No.1 **(Update)**							
Flowing 1	46	No Data	No Data	49	34	15	0
1A	56	50	6	52	49	3	-
1B	70	53	17	68	53	15	2
Hydrant Test No.2							
Flowing 2	86	74	12	86	77	9	3
2A	85	78	7	86	78	8	-1
2B	81	75	6	81	73	8	-2
2C	Not recorded	-	-	-	-	-	-
Hydrant Test No.3							
Flowing 3	74	No Data	No Data	75	57	18	-
3A	81	80	1	78	76	1	0
3B	85	85	0	86	85	1	-1
3C	74	68	6	75	70	6	0
Hydrant Test No.4							
Flowing 4	48	No Data	No Data	42	38	4	-
4A	64	60	4	59	56	3	1
4B	45	39	6	44	41	3	3
Hydrant Test No.5							
Flowing 5	72	No Data	No Data	73	72	1	-
5A	71	69	2	68	66	1	1
5B	70	69	1	70	69	0	1
5C	74	75	-1	77	77	1	-2
Hydrant Test No.6							
Flowing 6	84	No Data	No Data	83	73	10	-
6A	87	84	3	82	82	1	2
6B	91	89	2	91	90	1	1
Hydrant Test No.7							
Flowing 7	102	No Data	No Data	106	89	17	-
7A	110	108	2	107	104	3	-1
7B	102	105	-3	107	90	17	-20
7C	108	106	2	106	101	4	-3
Hydrant Test No.8							
Test No. 8 was not performed							
Hydrant Test No.9							
Flowing 9	90	No Data	No Data	94	74	20	-
9A	98	90	8	95	82	13	-5
9B	97	85	12	95	82	13	-1
9C	84	78	6	82	78	4	2
Hydrant Test No.10 **(Backwash/WTP Pumps Off)**							
Flowing 10	70	No Data	No Data	73	66	6	-
10A	72	60	12	72	68	5	7
10B	66	63	3	73	69	4	-1
10C	66	62	4	74	69	5	-1
Hydrant Test No.11 **(WTP Pumps Off)**							
Flowing 11	90	No Data	No Data	88	71	17	-
11A	90	80	10	89	74	15	-5
11B	82	74	8	87	72	16	-8
Hydrant Test No.12 **(WTP Pumps Off)**							
Flowing 12	52	No Data	No Data	56	52	3	-
12A	56	51	5	52	50	2	3
12B	57	52	5	55	52	3	2
12C	55	55	0	57	54	3	-3
Hydrant Test No.13							
Flowing 13	66	No Data	No Data	65	57	8	-
13A	69	59	10	66	58	8	2
13B	65	51	14	65	57	8	6
13C	65	55	10	65	57	8	2
Hydrant Test No.14							
Flowing 14	50	No Data	No Data	52	45	7	-
14A	62	51	11	55	48	7	4
14B	58	45	13	58	51	7	6

Table 5-3. Summary of Hydrant Test Calibration Results							
Hydrant	Field Data			Modeled Data			Comparison of Differential Pressures (Field - Model)
	Static Pressure, psi	Residual Pressure, psi	Differential Pressure, psi (Static - Residual)	Static Pressure, psi	Residual Pressure, psi	Differential Pressure, psi (Static - Residual)	
Hydrant Test No.15							
Flowing 15	58	No Data	No Data	54	43	11	-
15A	74	62	12	66	59	7	5
15B	63	51	12	64	56	8	4
15C	56	45	11	58	51	7	4
Hydrant Test No.16							
Flowing 16	82	No Data	No Data	81	72	9	-
16A	82	69	13	81	72	9	4
16B	86	71	15	86	77	9	6
16C	85	75	10	85	76	9	1
Hydrant Test No.17							
Flowing 17	66	No Data	No Data	58	44	14	-
17A	61	44	17	57	44	13	5
17B	59	44	15	60	49	10	5
Hydrant Test No.18							
Flow was not recorded during this test							

No adjustment in pipeline C-factors is recommended since all pipelines in this area are PVC or DI and the C-factors have been calibrated in other tests. It is recommended that the City checks the accuracy of the pressure gauges used for hydrant testing to ensure that they are correctly calibrated for future use.

5.5 SUMMARY OF FINDINGS AND CONCLUSIONS

Results from the hydrant test simulations indicate that the hydraulic model is generally well-calibrated using the pipeline C-factors shown in Table 5-4. The C-factor for cast iron pipelines less than 12-inches in diameter was changed from 75 to 90. All other pipeline C-factors remain unchanged.

Table 5-4. Calibrated Pipeline Roughness C-Factors Assigned in Hydraulic Model

Pipeline Material Type	Acronym	Hazen-Williams C-factor	
		Diameter < 12-inches	Diameter ≥ 12-inches
Cast Iron	CI	90	100
Ductile Iron	DI	130	140
Galvanized Steel	GALV	120	
Polyvinyl Chloride	PVC	140	
Steel	STL	120	
Unknown	UNK	120	

The results described in this section indicate that the City's water distribution system hydraulic model is adequate for use as a planning tool and can accurately simulate a fire flow or other large demand condition in the City's water system. It is recommended that the City: 1) check the accuracy of the pressure gauges used during hydrant testing; 2) verify the status of valves in the field, as identified in Hydrant Tests #1 and #13; and 3) continue to update the pipelines in the hydraulic model as facilities are constructed or replaced.

CHAPTER 6
Water System Analysis

This chapter presents an analysis of the City's existing water system and its ability to meet recommended water service and performance standards under future demands for the 20-year master plan horizon. The analysis includes both system capacity and hydraulic performance evaluations based on the performance criteria presented in *Chapter 4 Design and Performance Criteria*. The system capacity evaluation includes an evaluation of existing supply, pumping, and storage capacity for existing and projected water demand conditions. The performance evaluation assesses the water system's ability to meet recommended performance standards under future maximum day demand plus fire flow and future peak hour demand conditions.

The following sections present the evaluation methodology and results from the water system analysis:

- Existing Water System
- Future Water System
- Summary of Recommended Improvements

6.1 EXISTING WATER SYSTEM

The evaluation of the City's existing water system includes a system capacity evaluation of supply, pumping, and storage capacity. Evaluations, findings, and recommendations for addressing any deficiencies identified in the City's existing water distribution system are included in the following subsections. These recommendations are used to develop and prioritize a recommended CIP, which is further described in *Chapter 9 Capital Improvement Program.*

6.1.1 Existing Water Demands by Pressure Zone

Table 6-1 summarizes existing water demands by pressure zone. Water demands were spatially allocated into the hydraulic model using the annual metered water consumption data from 2020. The spatially located demands were then scaled to a total system average day demand of 0.85 mgd to match the annual average of total water produced in 2020. Maximum day and peak hour demands were calculated based on the adopted peaking factors of 2.4 and 3.6 times the average day demand, respectively, as described in *Chapter 3 Water Demand*.

Table 6-1. Existing Water Demands by Pressure Zone[a]						
Pressure Zone	Average Day Demand		Maximum Day Demand[b]		Peak Hour Demand[c]	
	gpm	mgd[d]	gpm	mgd[d]	gpm	mgd[d]
Main[e]	586	0.84	1,353	1.95	2,010	2.89
Strawberry	3	0.01	8	0.01	12	0.02
LakePointe	1	0.002	3	0.004	4	0.01
Subtotal (City)	552	0.80	1,326	1.91	1,988	2.87
WWTP	38	0.05	38	0.05	38	0.05
Total	**590**	**0.85**	**1,364**	**1.96**	**2,026**	**2.92**

(a) Demands spatially allocated based on 2020 water meter consumption data and scaled to match 2020 water production.
(b) MDD calculated using a peaking factor of 2.4 times the average day demand (see note (e)).
(c) PHD calculated using a peaking factor of 3.6 times the average day demand (see note (e)).
(d) Values shown are rounded to the nearest hundredth million gallon.
(e) The Main Zone MDD and PHD were calculated assuming MDD and PHD peaking factors of 1.0 for the WWTP.

6.1.2 Existing Water Facility Capacity Analysis

This section summarizes the evaluation of the City's existing supply, pumping, and storage capacity under existing water demand conditions.

6.1.2.1 Existing Supply Capacity Evaluation

The City's water supply is provided by local surface water diverted from the South Santiam River, which is impounded at the Foster Reservoir, and Ames Creek and treated at the City's WTP, as described in *Chapter 2 Existing System Description*. The City's water supply and treatment capacity criterion requires the City to produce sufficient supply to meet existing maximum day demand. The following sections evaluate the supply capacity of the City's water rights and WTP.

6.1.2.1.1 Water Rights Capacity Evaluation

The City holds existing water rights to the South Santiam River and Ames Creek, a tributary of the South Santiam River. At the time of this WMP the City does not divert water from Ames Creek. Therefore, it is excluded from this evaluation. The City holds three existing water rights for the South Santiam River which are summarized in Table 6-2. The water rights capacity evaluation presented in Table 6-2 is separated into permitted and certified water rights because Permit No. S-49959 is not fully perfected and is limited to 2.27 mgd. The City must demonstrate beneficial use of the remaining water right quantity of 1.28 mgd by 2050 to fully perfect Permit No. S-49959.

As shown in Table 6-2 the City's total existing certified water rights compared to the existing maximum day demand results in a total water rights capacity surplus of 5.22 mgd.

Table 6-2. Comparison of Available Water Rights and Required Supply Capacity, Existing Conditions					
Existing Water Right		Maximum Water Supply Capacity (Permitted)[a]		Maximum Water Supply Capacity (Certified)[a]	
Permit No.	Certificate No.	gpm	mgd	gpm	mgd
S-13151	88300	269	0.39	269	0.39
S-20525	88301	3,142	4.52	3,142	4.52
S-49959	88302	2,468	3.55	1,575	2.27
Total		**5,879**	**8.46**	**4,986**	**7.18**
Required Supply Capacity[b]		1,364	1.96	1,364	1.96
Total Existing Water Rights Capacity Surplus (Deficit)		**4,515**	**6.50**	**3,622**	**5.22**

(a) Permitted and certified water rights are shown in Table 2-1.
(b) Required supply capacity is equal to the existing maximum day demand (see Table 6-1).

6.1.2.1.2 Water Treatment Capacity Evaluation

As presented in *Chapter 2 Existing System Description*, the City's WTP has three parallel water treatment units, each with a nominal capacity of 1,400 gpm, for a total treatment capacity of 4,200 gpm, or approximately 6.0 mgd, and a firm capacity of 4.0 mgd, assuming a fully redundant filter. As shown in Table 6-3, the City's firm treatment capacity available at the WTP can supply the existing maximum day demand of 1.96 mgd. Therefore, no improvements are recommended to increase water treatment capacity.

Table 6-3. Available Water Treatment Capacity versus Existing Required Supply Capacity		
Water Treatment Component	Maximum Water Treatment Component Capacity	
	gpm	mgd
Treatment Unit #1	1,400	2.02
Treatment Unit #2	1,400	2.02
Treatment Unit #3	1,400	2.02
Total Capacity	**4,200**	**6.06**
Firm Capacity	**2,800**	**4.04**
Required Supply Capacity[a]	1,364	1.96
Total Existing Supply Capacity Surplus (Deficit)[b]	**1,436**	**2.08**

(a) Required supply capacity is equal to the existing maximum day demand (see Table 6-1).
(b) Capacity surplus calculated comparing firm capacity to required capacity.

6.1.2.2 Existing Pumping Capacity Evaluation

The City currently operates three pump stations, including the finished water pumps at the WTP, that serve to lift water into higher pressure zones.[1] The pumping capacity criterion for the City, described in *Chapter 4 Design and Performance Criteria*, requires the City's water system to provide sufficient pumping capacity to meet demands during normal operations. Normal operating conditions are defined as follows:

- For pump stations that serve a pressure zone with adequate gravity storage – Provide firm pumping capacity equal to maximum day demand for the pressure zone and all supported pressure zones
- For pump stations that serve a pressure zone with no gravity storage – Provide firm pumping capacity equal to the greater of: (1) peak hour demand; or, (2) maximum day demand plus fire flow

Firm pumping capacity assumes a reduction in total pumping capacity to account for pumps that are out of service at any given time due to mechanical breakdowns, routine maintenance, other operational problems, or water quality issues. At each pump station, firm pumping capacity is defined as the total pump station capacity with the largest pump out of service, and therefore not counted towards the overall total. Pump stations with only one pump have no firm capacity.

Table 6-4 compares the existing firm pumping capacity to the required existing pumping capacity for each pressure zone. The pumping capacity analysis indicates that the Main Zone and the Strawberry Zone have adequate firm pumping capacity to meet the City's pumping criterion under existing demand conditions. The LakePointe Pump Station (PS) does not have sufficient firm pumping capacity to provide the maximum day demand plus fire flow to the LakePointe Pressure Zone. Because the maximum day demand in the pressure zone is minimal, the LakePointe PS is deficient due to the required fire flow for single family residential land use (1,500 gpm). It is recommended that an additional 660 gpm of additional firm capacity be added to the LakePointe PS by upsizing existing pumps or adding additional pumps.

[1] The WTP finished water pumps are housed inside the WTP. For the purposes of this evaluation, they are referred to collectively as a pump station.

Table 6-4. Comparison of Available Pumping Capacity and Required Pumping Capacity, Existing Conditions, gpm

Pressure Zone	Pumping Facility	Pump ID / Serial Number	Pump Design Flow	Available Pumping Capacity, gpm		Required Pumping Capacity[a], gpm		Pumping Capacity Surplus (Deficit)
				Total Capacity	Firm Capacity	Criterion	Required Capacity	
Main	WTP Finished Water Pumps	161886	1400	4,200	2,800	MDD	1,353	1,447
		161887	1400					
		161888	1400					
Strawberry	Strawberry	Unknown	100	200	100	MDD	8	92
		Unknown	100					
LakePointe	LakePointe	Unknown	100	1,500	850	MDD + Fire	1,503	(653)
		Unknown	100					
		Unknown	650					
		Unknown	650					

(a) Required pumping capacity for zones with adequate storage is equal to the maximum day demand for the pressure zone, while zones without adequate storage require pumping capacity equal to the greater of peak hour demand or maximum day demand plus fire flow, as defined in Chapter 4 Design and Performance Criteria. Demands by zone are shown in Table 6-1

6.1.2.3 Existing Storage Capacity Evaluation

The City has four active water storage reservoirs, providing a total water system storage capacity of 4.31 MG.[2] To comply with the design and operational criteria, the water system should provide: 1) adequate operational storage to balance differences in demands and supplies; 2) emergency storage in case of supply failure; and, 3) water to fight fires. The City's available above-ground storage (i.e., storage reservoirs) must have sufficient capacity to meet the City's operational, emergency, and fire flow storage criteria.

The City's water storage capacity requirement is described in Chapter 4, and is described as follows:

- Operational storage equal to 25 percent of maximum day demand
- Emergency storage equal to two maximum day demands
- Fire flow storage equal to the highest fire flow and duration recommended in the pressure zone

The City's existing water storage facilities were evaluated to determine whether the City's existing water system has sufficient storage capacity to provide the recommended operational, emergency, and fire flow storage. Table 6-5 compares the City's available water storage capacity with the existing required storage capacity by pressure zone. As shown, the City does not have sufficient storage capacity to meet the required storage capacity criteria in either the Main Zone, where 1.5 MG additional storage is required, or the Strawberry Zone, where 0.1 MG of additional storage is required.

The need for additional storage in the City's water system confirms concerns from City staff, especially in the event of a rolling blackout or other emergency that could require the system to be served only by gravity storage for an extended period. It is recommended that the City construct additional gravity storage to serve the Main Zone to address the existing storage deficit. The Strawberry Zone already has a large volume of storage with respect to the demands in the zone, and consequently the City has difficulty maintaining disinfectant residuals in the Strawberry Reservoir. Additional storage is not recommended for the Strawberry Zone. However, the City should make pipeline improvements to improve conveyance capacity and ensure that the required fire flow and volume in the pressure zone can be met by a combination of storage, pumping, and an existing check-valve connection with the Main Zone.

[2] A fifth reservoir, the 300k gal 10th Ave Reservoir is currently offline due to severe cracking in the foundation, and corresponding water loss. The City does not currently have plans to reactivate the reservoir.

Table 6-5. Comparison of Available Storage Capacity and Required Storage Capacity, Existing Conditions

Pressure Zone(s)	Available Storage Capacity, kgal			Required Storage Capacity, kgal				Storage Surplus (Deficit), kgal
	Storage Facility	Capacity	Zone Storage	Operational[a]	Emergency[b]	Fire[c]	Total	
Main[d]	10th Ave - 300K (Offline)	300	4,200	0	0	1,320	1,320	2,880
	10th Ave - 700K	700						
	10th Ave - 1.5M	1,500						
	49th Ave	2,000						
Strawberry	Strawberry	110	110	0	0	180	180	(70)

(a) Operational storage capacity is equal to 25 percent of the maximum day demand of the zone and all zones supported solely by pumping from that zone. See Table 6-1 for projected maximum day demand.

(b) Emergency storage capacity is equal to one average day demand of the zone plus all zones supported solely by pumping from that zone. See Table 6-1 for projected average day demand.

(c) Fire flow storage capacity required is equal to the largest fire flow possible in zone: 5,500 gpm for 4 hours for the Main Zone; 1,500 gpm for 2 hours in all other zones.

(d) The LakePointe zone is supplied solely by the Main zone via pumping. The Main zone was evaluated using the total operational and emergency requirements of both pressure zones.

6.2 FUTURE WATER SYSTEM

The evaluation of the City's future water system includes a system capacity evaluation that builds upon the existing system evaluation. Evaluations, findings, and recommendations for addressing any deficiencies identified in the City's future water distribution system are included in the following subsections. These recommendations are used to develop and prioritize a recommended CIP, which is further described in *Chapter 9 Capital Improvement Program.*

6.2.1 Future Water System Facility and Network Assumptions

Initial discussions of proposed water system improvements with the City indicated the need for major system configuration changes. Figure 6-1 shows the future system configuration used to capture the City's operational goals, and appropriately size facilities. This configuration is the basis for the future system capacity evaluation. The key proposed changes to the City's system are summarized in the following paragraphs.

6.2.1.1 Improvements in Main Pressure Zone

High pressures, greater than 100 psi, are experienced in much of the Main Pressure Zone under normal operating conditions. These high pressures are exacerbated when the City operates the WTP finished water pumps to fill the Main Zone reservoirs. The City does not operate more than one finished water pump at a time. Additionally, the 10th Ave Reservoirs located at the southwest end of the City are more hydraulically distant from the WTP than the 49th Ave Reservoir, causing the 49th Ave Reservoir to fill significantly faster if flow to the reservoir is uncontrolled. The City currently restricts flow to the 49th Ave Reservoir by partially closing a valve on the inflow/outflow pipeline to the reservoir. The proposed improvements to mitigate these issues are:

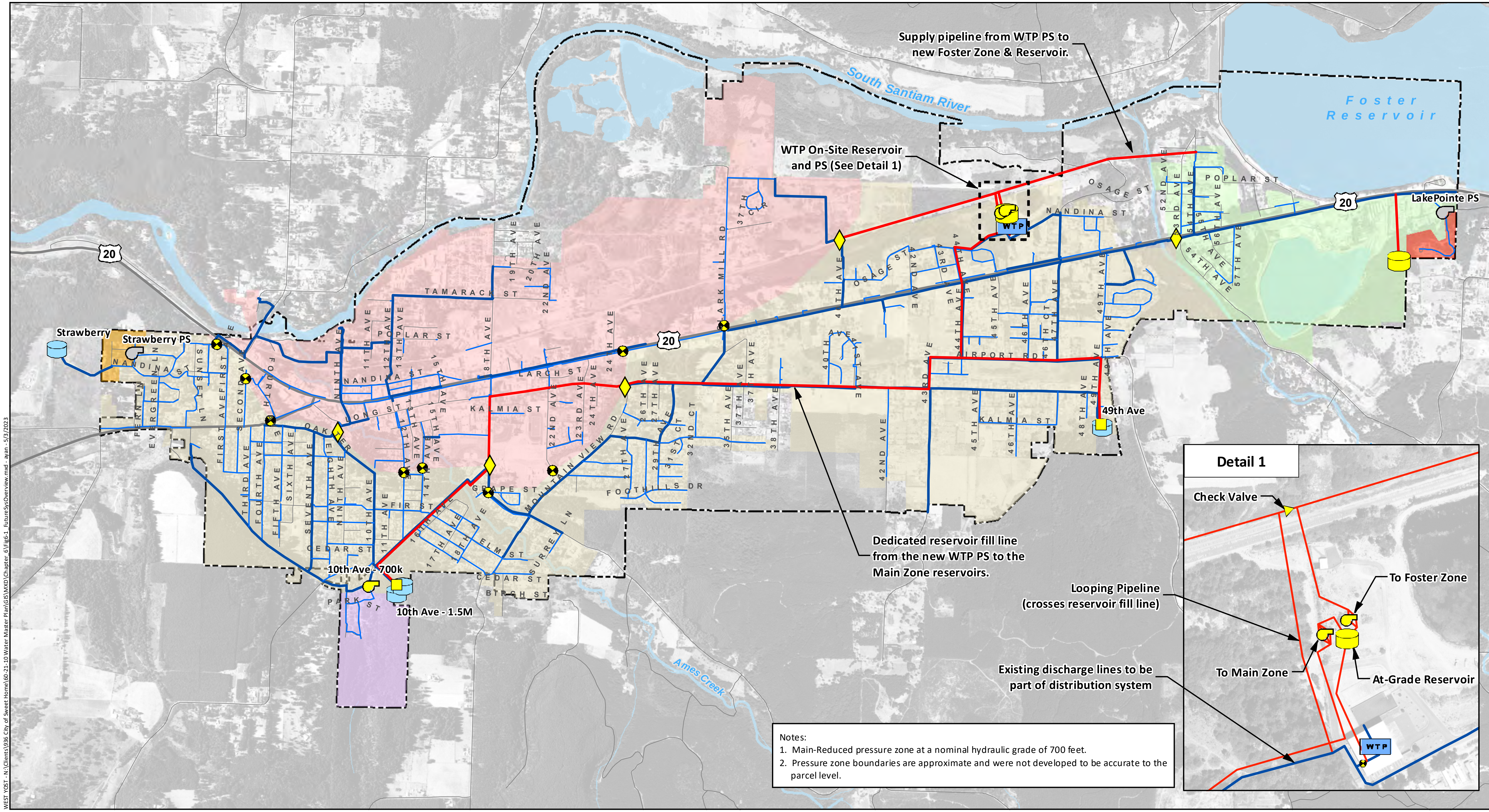
1. Reconfigure the Main Zone to supply the lower elevation areas of the pressure zone via PRV's and alleviate high pressures (identified in Figure 6-1 as the Main-Reduce Zone);
2. Install an at-grade finished water reservoir at the WTP with a pump station to pump into the Main Zone;
3. Install a dedicated transmission pipeline direct from the new WTP pump station to the Main Zone reservoirs to simplify reservoir operations; and,
4. Install altitude valves at the Main Zone reservoirs to further control reservoir levels.

6.2.1.2 Improvements East of Wiley Creek

The City is concerned with its ability to reliably serve customers east of Wiley Creek and south of the Foster Reservoir. This area is supplied from the Main Zone solely by a 16-inch pipeline crossing over Wiley Creek, which is a single point of failure to this service area (there is no existing storage east of the Wiley Creek crossing). The proposed improvements to mitigate this issue are:

1. Construct a storage reservoir sited in the undeveloped hills immediately west of the LakePointe Zone; and,
2. Install pumps at the new WTP pump station to fill the new reservoir and a new supply pipeline parallel to the existing railroad north of the WTP.

This new pressure zone is identified in Figure 6-1 as the Foster Zone.



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Recommended Pressure Zones
- Strawberry
- LakePointe
- Main
- Main-Reduced (New)
- Foster (New)
- 10th Ave (New)

- Existing Water Treatment Plant
- Existing Storage Tank
- Potable Water Pump Station

Existing System Pipelines
- Diameter Less than 10-inches
- Diameter 10-inches and Greater

- Recommended Pump Station
- Recommended Storage Tank
- Recommended Normally Closed Valve
- Recommended Altitude Valve
- Recommended Pressure Reducing Valve

- Required New Pipeline for Recommended Operations
- City Limit

N

0 900 1,800

Scale in Feet

DRAFT

Figure 6-1

Operational Overview of Recommended Future System

City of Sweet Home
Water Master Plan

6.2.1.3 Improvements to Address Low Pressures

The City currently experiences unacceptably low pressures in the area immediately west and southwest of the 10th Ave Reservoirs. The proposed improvement to mitigate this issue is a new pump station sited near southern terminus of 10th Ave which would supply a new closed pressure zone. This new pressure zone is identified in Figure 6-1 as the 10th Ave Zone.

The improvements described above were the basis for the facility capacity evaluations presented in Section 6.1.2. The proposed Foster and 10th Ave pressure zones are included in subsequent tables so that the facilities proposed to serve these pressure zones could be appropriately sized for the demands and land uses in each pressure zone.

6.2.2 Projected Water Demands by Pressure Zone

Table 6-6 summarizes future water demands summarized by pressure zone. The total 2043 system average day demand of 1.10 mgd corresponds to the sum of existing water demands (0.85 mgd) and projected new water demand (0.25 mgd). Maximum day and peak hour demands were calculated based on the adopted peaking factors of 2.4 and 3.6 times the average day demand, respectively, as described in *Chapter 3 Water Demand*.

Table 6-6. Future Water Demands by Pressure Zone[a]

Pressure Zone	Average Day Demand		Maximum Day Demand[b]		Peak Hour Demand[c]	
	gpm	mgd[d]	gpm	mgd[d]	gpm	mgd[d]
Main / Main Reduced (New)[e]	716	1.03	1,664	2.40	2,478	3.57
Strawberry	4	0.01	9	0.01	14	0.02
LakePointe	2	0.003	6	0.008	9	0.01
Foster (New)	29	0.04	71	0.10	106	0.15
10th Ave (New)	12	0.02	30	0.04	45	0.07
Subtotal (City)	726	1.05	1,742	2.51	2,613	3.77
WWTP	38	0.05	38	0.05	38	0.05
Total	**764**	**1.10**	**1,780**	**2.56**	**2,651**	**3.82**

(a) Future water demands are equal to existing water demands (refer to Table 6-1) plus new water demand projected by 2043. The distribution of new water demand is discussed in Section 6.2.4.
(b) Maximum day demand (MDD) calculated using a peaking factor of 2.4 times the average day demand (see note (e)).
(c) Peak hour demand (PHD) calculated using a peaking factor of 3.6 times the average day demand (see note (e)).
(d) Values are rounded to the nearest hundredth million gallon.
(e) The Main-Reduced Zone MDD and PHD were calculated assuming a 1.0 MDD and PHD peaking factor for the WWTP.

As discussed in *Chapter 3 Water Demand* and as shown in Figure 3-2, the projected water demand was proportionally distributed among the City's future development areas. The projected water demand for each development area was assigned to the demand node closest to the associated development area in the hydraulic model.

6.2.3 Future Water Facility Capacity Analysis

This section summarizes the evaluation of the City's existing supply, pumping, and storage capacity under future water demand conditions. The evaluations build upon those presented in Section 6.1.2.

6.2.3.1 Future Supply Capacity Evaluation

The following sections evaluate the supply capacity of the City's water rights and water treatment facility when compared to future 2043 water demands.

6.2.3.1.1 Water Rights Capacity Evaluation

Table 6-7 presents the results of the future water rights capacity evaluation. The City's total existing certified water rights compared to the future maximum day demand results in a total water rights capacity surplus of 4.62 mgd.

Table 6-7. Comparison of Available Water Rights and Required Supply Capacity, Future Conditions

Existing Water Right		Maximum Water Supply Capacity (Permitted)[a]		Maximum Water Supply Capacity (Certified)[a]	
Permit No.	Certificate No.	gpm	mgd	gpm	mgd
S-13151	88300	269	0.39	269	0.39
S-20525	88301	3,142	4.52	3,142	4.52
S-49959	88302	2,468	3.55	1,575	2.27
Total		**5,879**	**8.46**	**4,986**	**7.18**
Required Supply Capacity[b]		1,780	2.56	1,780	2.56
Total Existing Water Rights Capacity Surplus (Deficit)		**4,099**	**5.90**	**3,206**	**4.62**

(a) Permitted and certified water rights are shown in Table 2-1.
(b) Required supply capacity is equal to the projected maximum day demand (see Table 6-6).

6.2.3.1.2 Water Treatment Capacity Evaluation

Table 6-8 presents the results of the future water treatment capacity evaluation. As shown in Table 6-3, the City's treatment capacity available at the WTP can supply the future maximum day demand of 2.56 mgd. Therefore, no improvements are recommended to increase water treatment capacity.

Table 6-8. Available Treatment Capacity versus Future Required Supply Capacity		
Water Treatment Component	Maximum Water Treatment Component Capacity	
	gpm	mgd
Treatment Unit #1	1,400	2.02
Treatment Unit #2	1,400	2.02
Treatment Unit #3	1,400	2.02
Total Capacity	**4,200**	**6.06**
Firm Capacity	**2,800**	**4.04**
Required Supply Capacity[(a)]	1,780	2.56
Total Existing Supply Capacity Surplus (Deficit)[(b)]	**1,020**	**1.48**

(a) Required supply capacity is equal to the projected maximum day demand (see Table 6-6).
(b) Capacity surplus calculated comparing firm capacity to required capacity.

6.2.3.2 Future Pumping Capacity Evaluation

Table 6-9 compares the existing firm pumping capacity to the required future pumping capacity for each pressure zone. The Main, Foster, and 10th Ave pressure zones were evaluated with no existing available pumping capacity because the City does not currently have infrastructure to serve these zones.[3]

As shown in Table 6-9, the Strawberry Zone is the only pressure zone in the future water system with a pumping supply capacity surplus. The LakePointe Zone is projected to experience minimal growth in water demand by 2043, and the firm pumping capacity deficit of approximately 660 gpm represents no significant change compared to the existing firm pumping capacity deficit (see Table 6-4). The Main-Reduced pressure zone would require approximately 1,700 gpm of firm pumping capacity, and the Foster Zone would require approximately 80 gpm, to meet the City's pumping capacity criterion. As shown on Figure 6-1, it is recommended that pumping capacity for both the Main and Foster zones would be sited at the WTP in a single dual-zone pump station. Lastly, the 10th Ave Zone would require approximately 1,530 gpm of total firm pumping capacity to meet the City's pumping capacity criteria: 30 gpm of firm pumping capacity to provide the MDD and 1,500 gpm to provide fire flow to the single family homes in the zone.

6.2.3.3 Future Storage Capacity Evaluation

Table 6-10 compares the City's available water storage capacity with the future required storage capacity by pressure zone. As shown, the City does not have sufficient storage capacity to meet the required storage capacity criteria in any pressure zone. The Strawberry Zone experiences a deficit of 0.1 MG under future demand conditions, similar to existing demand conditions. While a portion of the Main Zone is re-zoned to the new Foster Zone, Table 6-10 indicates a significant storage deficit of approximately 2.6 MG under future conditions. Furthermore, approximately 0.8 MG of storage is required to provide local gravity storage to the new Foster Zone.

[3] The analysis of the Main Zone includes the planned Main Reduced Zone, which would be served from the Main Zone.

Table 6-9. Comparison of Available Pumping Capacity and Required Pumping Capacity, Future Conditions, gpm

Pressure Zone	Pumping Facility	Pump Design Flow, gpm	Available Pumping Capacity, gpm		Required Pumping Capacity[a], gpm		Pumping Capacity Surplus (Deficit)
			Total Capacity	Firm Capacity	Criteria	Required Capacity	
Main / Main-Reduced (New)	WTP - Main Zone (New)	-	-	-	MDD	1,704	(1704)
Strawberry	Strawberry	100 100	200	100	MDD	9	91
LakePointe	LakePointe	100 100 650 650	1,500	850	MDD + Fire	1,506	(656)
Foster (New)	WTP - Foster Zone (New)	-	-	-	MDD	76	(76)
10th Ave (New)	10th Ave (New)	-	-	-	MDD + Fire	1,530	(1530)

(a) Required pumping capacity for zones with adequate storage is equal to the maximum day demand for the pressure zone, while zones without adequate storage require pumping capacity equal to the greater of peak hour demand or maximum day demand plus fire flow, as defined in Chapter 4 Design and Performance Criteria. Demands by zone are shown in Table 6-6.

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Table 6-10. Comparison of Available Storage Capacity and Required Storage Capacity, Future Conditions									
Pressure Zone(s)	Available Storage Capacity, kgal			Required Storage Capacity, kgal				Storage Surplus (Deficit), kgal
	Storage Facility	Capacity	Zone Storage	Operational[a]	Emergency[b]	Fire[c]	Total	
Main/ Main-Reduced (New)	10th Ave - 700K	700	4,200	0	0	1,320	1,320	2,880
	10th Ave - 1.5M	1,500						
	49th Ave	2,000						
Foster (New)[d]	-	-	-	0	0	540	540	(540)
Strawberry	Strawberry	110	110	0	0	180	180	(70)

(a) Operational storage capacity is equal to 25 percent of the maximum day demand of the zone and all zones supported solely by pumping from that zone. See Table 6-6 for projected maximum day demand.

(b) Emergency storage capacity is equal to one average day demand of the zone plus all zones supported solely by pumping from that zone. See Table 6-6 for projected average day demand.

(c) Fire flow storage capacity required is equal to the largest fire flow possible in zone: 5,500 gpm for 4 hours for the Main Zone; 1,500 gpm for 2 hours in all other zones.

(d) The LakePointe zone is supplied solely by the Foster zone via pumping. The Foster zone was evaluated using the total operational and emergency requirements of both pressure zones.

It is recommended that the identified storage deficits be mitigated through a single 3.0 MG storage reservoir at the WTP, and a single 0.8 MG reservoir in the Foster Zone shown in Figure 6-1. It should be noted that the proposed WTP PS must be equipped with adequate backup power (and fuel storage) to convey the storage volume at the WTP to the Main Zone, as it would not be sited at a hydraulic grade to serve the Main Zone by gravity in the event of a power failure (i.e., an emergency condition).

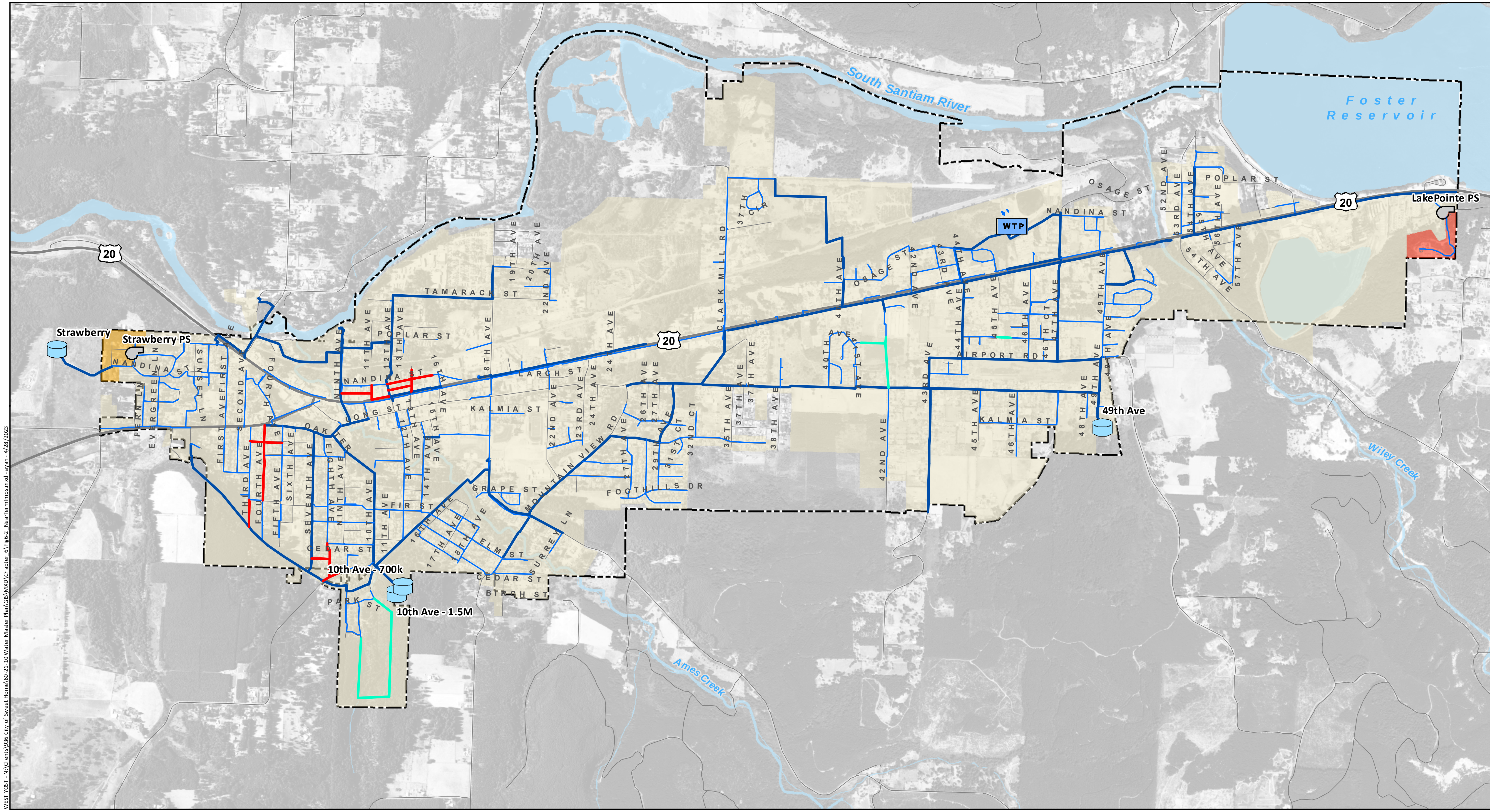
6.2.4 Future Water System Performance Analysis

The water system performance evaluation identifies necessary improvements to support the City's future water demands while meeting the City's recommended water system performance criteria.

The hydraulic model was updated to include the following ongoing and planned pipeline improvement projects, also shown on Figure 6-2:

- **Planned Pipeline Infrastructure Projects:** Identified near-term expansions/improvements; assuming these are already funded and in design/construction. These projects are not included in the recommended CIP, since they are already funded and are in design/construction.
- **Developer-Identified Improvements:** New looping to serve identified development projects. These projects are not included in the recommended CIP, since they and will be developer-funded.

The distribution system updated with the above improvements is referred to as the "existing distribution system." Subsequently, the hydraulic model was also updated to include all future system improvements described in Section 6.2.1 and shown in Figure 6-1.



Pressure Zones
- Strawberry
- LakePointe
- Main

WTP Existing Water Treatment Plant
Existing Storage Tank
Pump Station

Existing System Pipelines
- Diameter Less than 10-inches
- Diameter 10-inches and Greater

Ongoing Pipeline Improvement
Developer-Funded Pipeline Improvement
City Limit

Notes:
1. All pipeline improvements shown are 8-inch if looped and 6-inch if a dead-end.

0 900 1,800
Scale in Feet

DRAFT

Figure 6-2

Future Near-Term Pipeline Improvements

City of Sweet Home
Water Master Plan

Hydraulic evaluations were performed using the City's updated hydraulic model to assess the performance of the water distribution system under future water demand conditions, first for the existing distribution system to identify deficiencies, and then with the future water system configuration to identify any improvements needed in addition to reconfiguration improvements. The following scenarios were evaluated:

- **Normal Operations – Peak Hour Demand:** A peak hour flow condition was simulated for the distribution facilities to evaluate their capacity to meet the projected peak hour demand scenario. Peak hour demands are met by a combination of supply from storage reservoirs and pump stations.
- **Fire Flow Availability – Maximum Day Demand plus Fire Flow:** To evaluate the water system under the maximum day demand plus fire flow scenario, InfoWater®'s *"Available Fire Flow Analysis"* tool was used to determine the available fire flow while meeting the maximum day demand plus fire flow performance criteria within the water system. Additional improvements required specifically to meet fire flows were identified under this evaluation.

6.2.4.1 Peak Hour Demand

The peak hour demand scenario evaluates the hydraulics of the City's water system during a peak hour demand condition. An overview of the evaluation criteria and a discussion of the results are presented below.

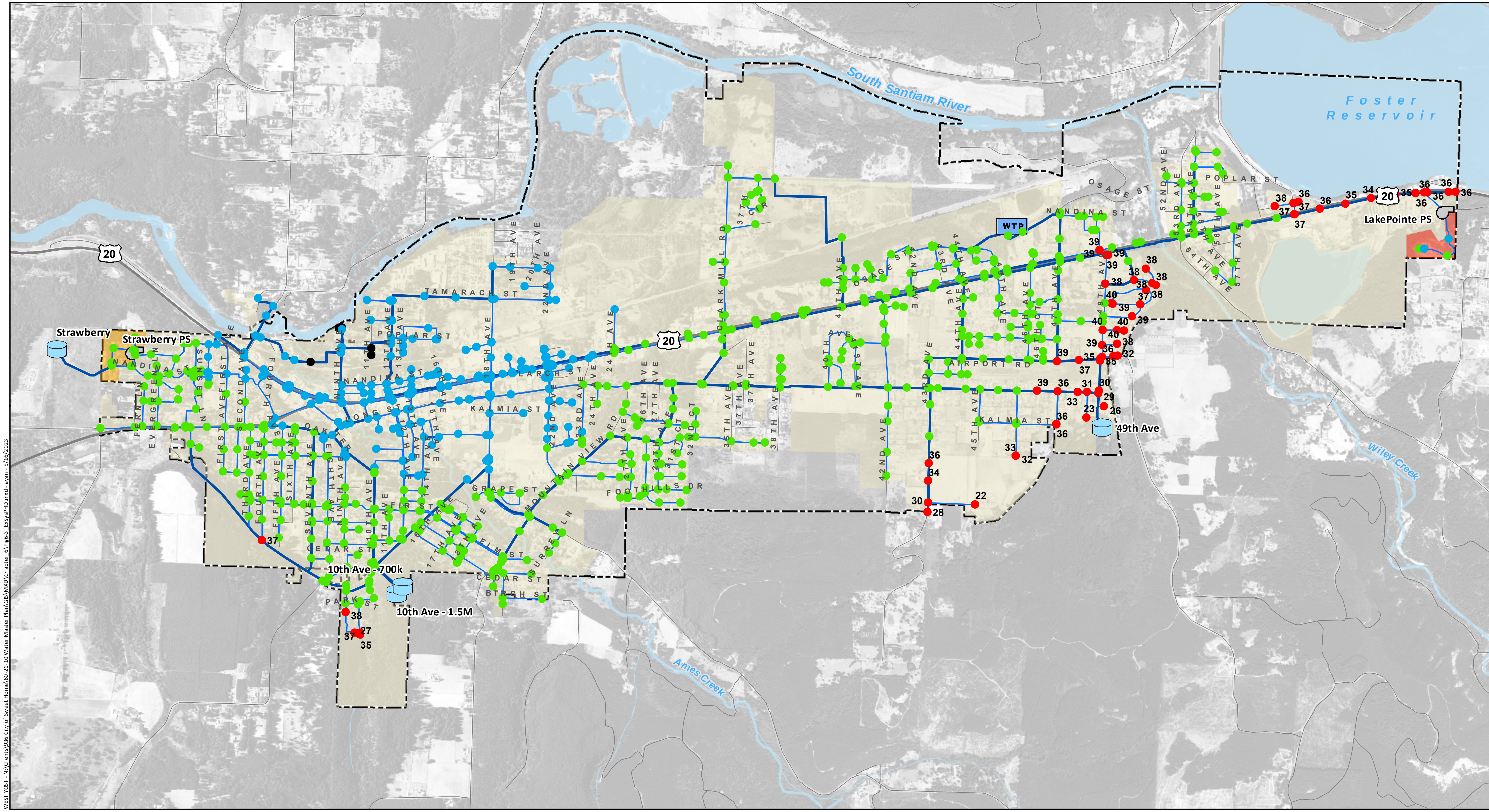
6.2.4.1.1 Evaluation Overview

The projected peak hour demand for the City is 2,651 gpm (3.82 mgd). The City's peak hour demand minimum pressure performance criterion requires that 40 psi be maintained throughout the water system under peak hour conditions. In addition, new pipelines should be designed such that velocities do not exceed 5 ft/s.

6.2.4.1.2 Evaluation Results

The City's existing water system is able to deliver peak hour demand while maintaining 40 psi at most locations within the City. The model results illustrated in Figure 6-3 show that high elevation areas of the Main-Zone to the north and southwest of the 49th Ave Reservoir experience pressures below 40 psi, with some dead ends below 30 psi. Similarly, low pressures below 40 psi are experienced along the Santiam Highway as it parallels Foster Reservoir, and the area southwest of the 10th Ave Reservoirs. High pressures above 80 psi are experienced in the northwest part of the existing Main Zone; pressures increase gradually moving south to north as elevation decreases.

These deficiencies reinforce the need for the major system configuration changes identified by the City, described in detail in Section 6.2.1, and shown on Figure 6-1.



Pressure

- Less than 20 psi
- 20 psi - 40 psi
- 40 psi - 80 psi
- 80 psi - 100 psi
- Greater than 100 psi

WTP Water Treatment Plant

Storage Tank

Pump Station

Existing System Pipelines

- Diameter Less than 10-inches
- Diameter 10-inches and Greater

City Limit

Pressure Zones

- Main
- Strawberry
- LakePointe

Notes:
1. Existing system pipelines include all existing pipelines, near-term pipeline improvements in design/construction, and identified developer-funded looping. Refer to Figure 6-2 for additional detail on the existing system network.
2. Existing system was evaluated under a future peak hour demand equal to 3.82 mgd (2,651 gpm). One WTP finished water pump and the LakePointe PS are online, and all other pumps are offline.
3. Black labels represent the system pressure. Only locations with a modeled pressure less than 20 psi are labeled.

0 900 1,800
Scale in Feet

WEST YOST

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Figure 6-3

Existing System
Future Peak Hour Demand

City of Sweet Home
Water Master Plan

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Figure 6-4 shows the future system pressures under future peak hour demand conditions, with all proposed improvements implemented. An altitude valve at the 49th Ave Reservoir, instead of the throttled valve on the inflow/outflow pipe, would boost pressures in the immediate area surrounding the 49th Ave Reservoir. A new storage reservoir and creation of the Foster Zone would improve pressure in the area east of Wiley Creek. Finally, strategic placement of PRVs and closed valves to create the Main-Reduced Zone would lower the majority of the high pressures shown in Figure 6-3 to be within a more desirable range (40 to 80 psi). However, some areas with pressures greater than 80 psi remain at the lower elevation areas of the new Main Zone boundary due to the placement of PRVs and normally closed valves to most feasibly isolate the Main-Reduced Zone.

It is worth noting that the 49th Ave Reservoir is sited too low to maintain pressures above 40 psi under peak hour conditions in some pipelines at the highest elevations in the vicinity of the reservoir, even with all recommended improvements. No infrastructure improvements are recommended to address this deficiency. The City normally operates the 49th Ave Reservoir level above 70 percent full to maintain a pressure range of 35 to 40 psi for customers. Additionally, the City owns and operates a small hydropneumatic pump station to serve the few high elevation customers in the vicinity of the reservoir.

6.2.4.2 Maximum Day Demand Plus Fire Flow

The maximum day demand plus fire flow scenario evaluates the fire flow availability in the City's water system under a future maximum day demand condition. Additional improvements were identified to meet the fire flow criteria outlined in *Chapter 4 Design and Performance Criteria*. An overview of the evaluation criteria and a discussion of the results are presented below.

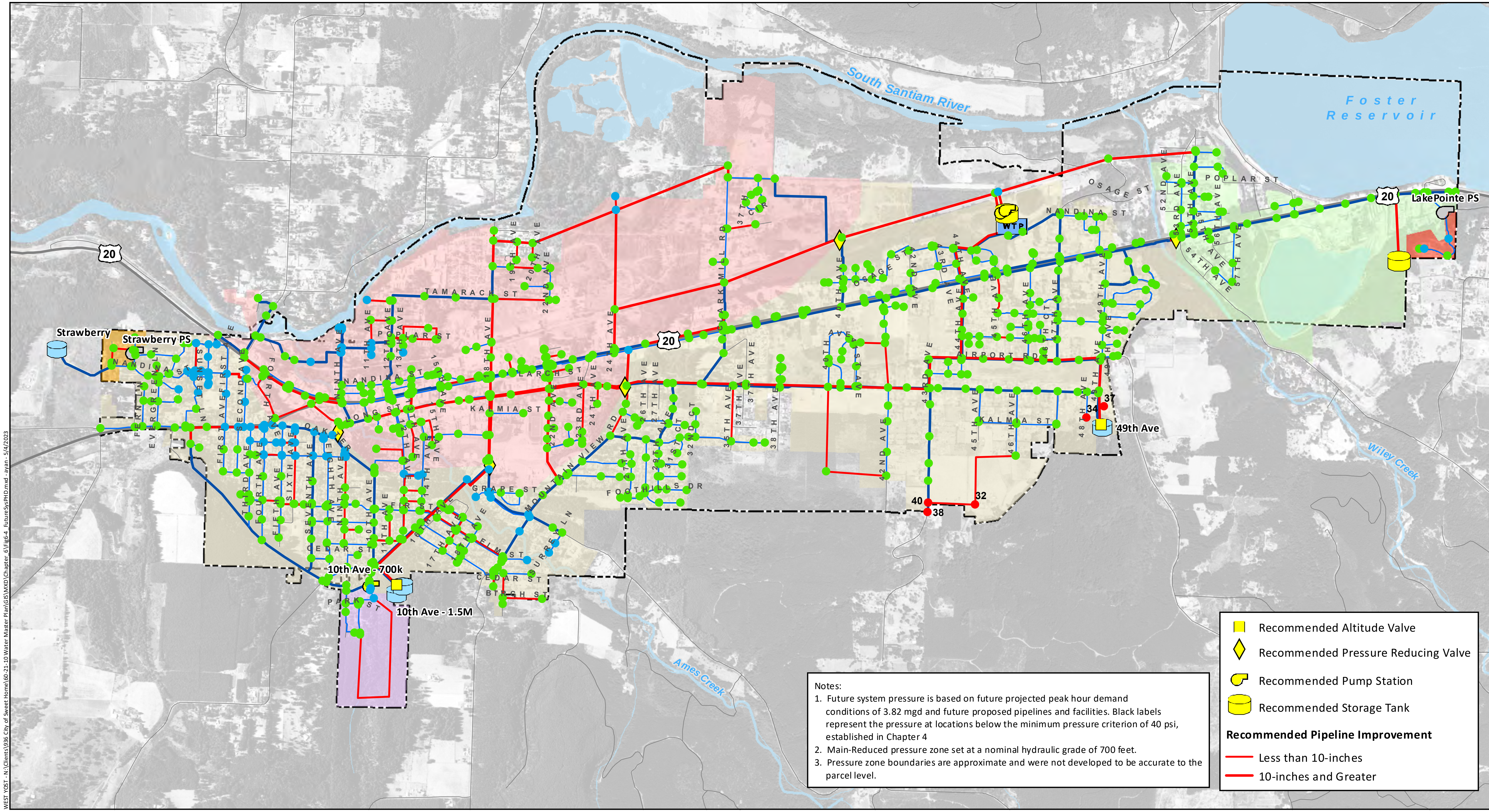
6.2.4.2.1 Evaluation Overview

A projected 2043 maximum day demand of 1,780 gpm (2.56 mgd) for the City was used for the evaluation. The City's minimum pressure criterion requires that a 20 psi residual pressure be maintained throughout the water system under maximum day demand plus fire flow. In addition, new pipelines should be designed such that velocities do not exceed 12 ft/s under fire conditions.

Fire flows were assigned to hydrant locations based on adjacent land use(s), per the City's Comprehensive Plan Land Use (amended in 2010) and fire flow requirements outlined in Chapter 4. Figure 6-5 shows the fire flow requirements assigned to hydrant locations. Generally, fire flow requirements are lower on the outskirts of the City and increase closer to the Santiam Highway and the adjacent commercial areas. It should be noted that manual adjustments were made to some fire flow requirements to better represent the building purpose and size. For example, hydrants adjacent to Sweet Home High School were assigned a fire flow of 5,500 gpm for 4 hours which is much higher than the surrounding land uses of Central Commercial (3,000 gpm for 3 hours).

6.2.4.2.2 Evaluation Results

Figure 6-6 shows the locations of deficient hydrants in the existing system under future maximum day demand conditions. A majority of the City's commercial and industrial areas, as well as schools, are deficient due to large fire flow requirements (3,000 gpm and greater). Many of the hydrants on 2-inch diameter pipelines, which are mostly located in the western half of the City, are deficient by greater than 1,000 gpm. Other areas of concern include long dead-end pipelines, areas with a single supply pipeline (i.e., the Foster Area east of Wiley Creek), and high-elevation areas.



Pressure
- 20 psi - 40 psi
- 40 psi - 80 psi
- 80 psi - 100 psi

Recommended Pressure Zones
- Foster
- Main
- Main-Reduced
- 10th Ave
- Strawberry
- LakePointe

WTP Existing Water Treatment Plant

Existing Storage Tank

Potable Water Pump Station

Existing System Pipelines
- Diameter Less than 10-inches
- Diameter 10-inches and Greater

City Limit

0 900 1,800
Scale in Feet

DRAFT

Figure 6-4
Recommended System Improvements Under Future Peak Hour Demand

City of Sweet Home
Water Master Plan

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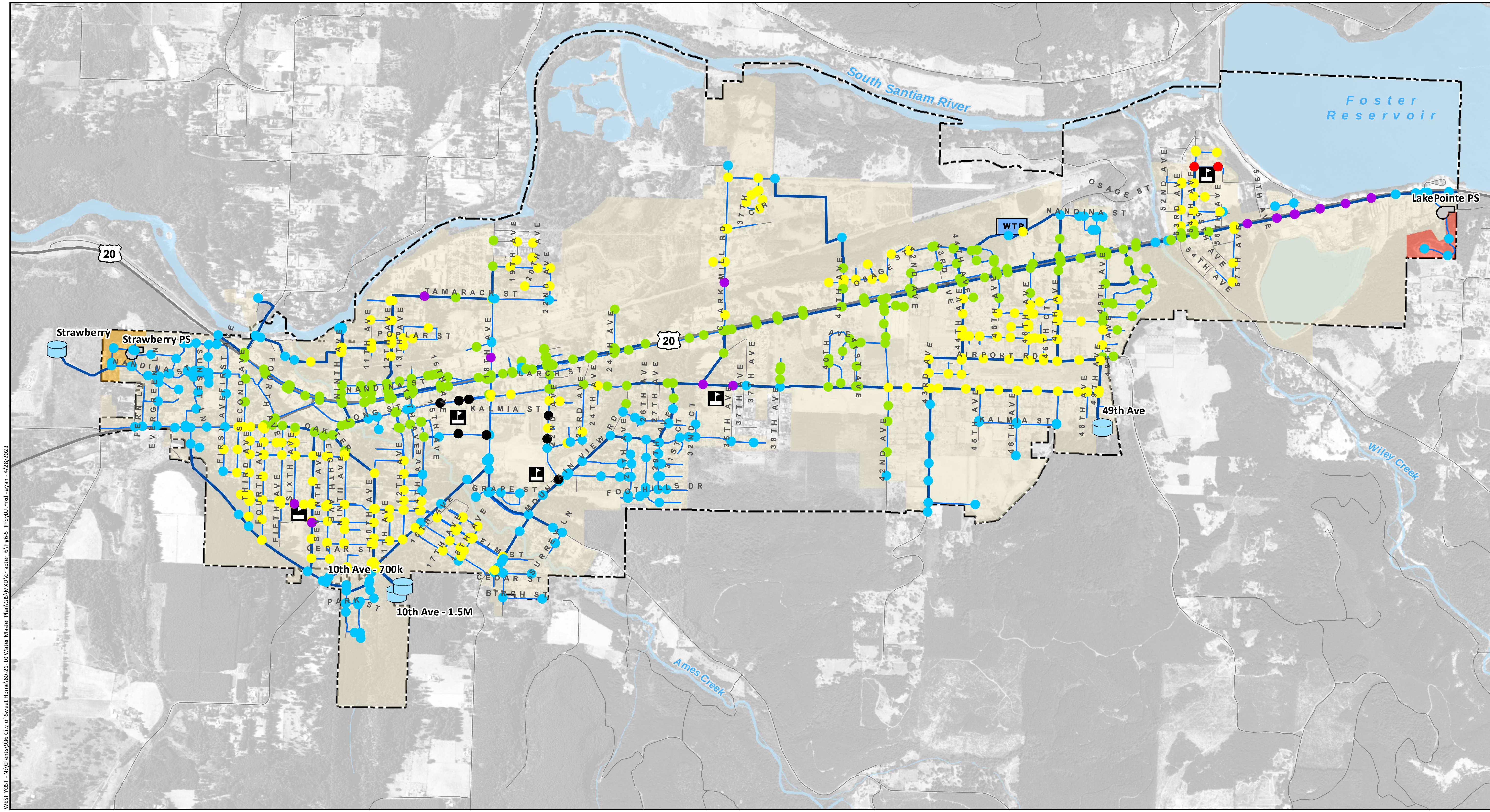

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Fire Flow Requirement

- 1,500 gpm
- 2,000 gpm
- 3,000 gpm
- 4,000 gpm
- 4,500 gpm
- 5,500 gpm

WTP Water Treatment Plant

Storage Tank

Pump Station

Existing System Pipelines

- Diameter Less than 10-inches
- Diameter 10-inches and Greater

City Limit

School

Pressure Zones

- Main
- Strawberry
- LakePointe

Notes:

1. Required fire flow was assigned at each hydrant based on land use from the Sweet Home Comprehensive Plan Zoning designation.

N

0 900 1,800

Scale in Feet

WEST YOST

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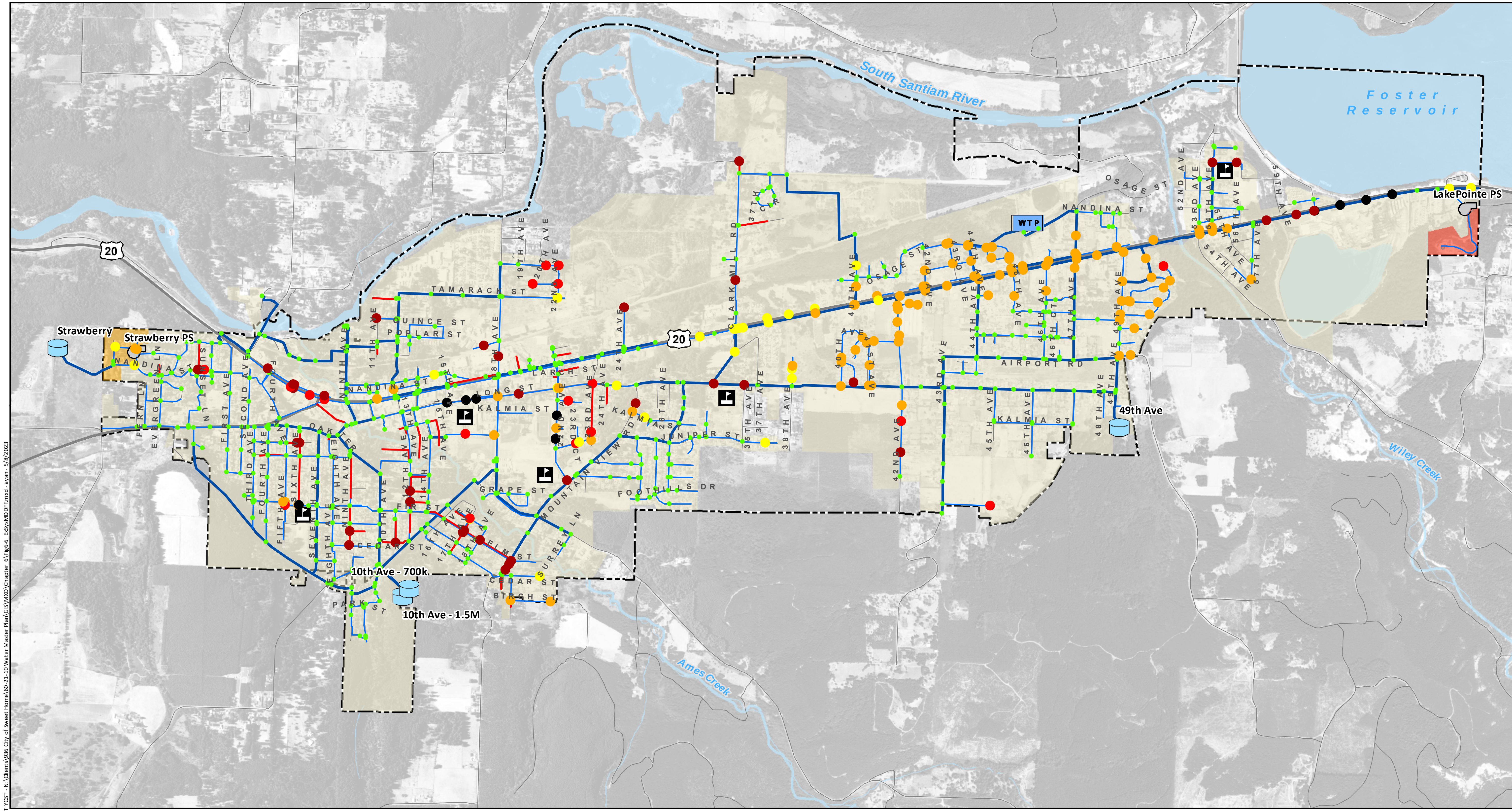
Figure 6-5

Required Fire Flow by Land Use

City of Sweet Home

Water Master Plan



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- Junction Meets Fire Flow Requirement

Flow to Meet Fire Flow Requirement at Deficient Junctions

- Less than 200 gpm
- 200 - 500 gpm
- 500 - 1,000 gpm
- 1,000 - 2,000 gpm
- Greater than 2,000 gpm

WTP Water Treatment Plant

Storage Tank

Pump Station

Existing System Pipelines

- 2-inches in Diameter
- Diameter 2-inches to 8-inches
- Diameter 10-inches and Greater

City Limit

School

Pressure Zones

- Main
- Strawberry
- LakePointe

Notes:

1. Existing system pipelines include all existing pipelines, near-term pipeline improvements in design/construction, and identified developer-funded looping. Refer to Figure 6-3 for additional detail on the existing system network.
2. Existing system was evaluated under a future maximum day demand equal to 2.56 mgd (1,780 gpm). One WTP finished water pump and the LakePointe PS are online, and all other pumps are offline.
3. Refer to Figure 6-5 for the required fire flow at each junction.

N

0 900 1,800

Scale in Feet

WEST YOST

DRAFT

Figure 6-6

Existing System
Future MDD - Fire Flow Availability

City of Sweet Home
Water Master Plan

Improvements identified to improve fire flow availability are generally described as:

1. Replacing all 2-inch pipelines with 6-inch (dead-ends) or 8-inch (looped) pipelines;
2. Replacing pipelines 8-inches or less in diameter with 10-inch or 12-inch pipelines in high flow areas (i.e., near schools); and,
3. Looping existing dead-ends or isolated areas with segments of new pipelines.

Figure 6-7 shows the locations of deficient hydrants with all recommended water system improvements. A majority of junctions now meet the City's fire flow requirement, though there are some locations throughout the City that are still deficient. These areas are predominantly located on dead-end pipelines with large fire flow requirements, or near schools with very high fire flow requirements. Each area was reviewed to determine if the deficiency warranted further pipeline improvements. All remaining deficiencies shown on Figure 6-7 do not warrant additional pipeline improvements (e.g., pipeline is relatively new, upsizing would result in an unreasonably large dead-end, etc.) or can be met by multiple fire hydrants. The required fire flows at schools (ranging from 4,000 gpm to 5,500 gpm) cannot be realistically provided by a single hydrant; rather, it was confirmed that the recommended pipeline improvements around schools are adequate to meet the required fire flow using multiple hydrants.

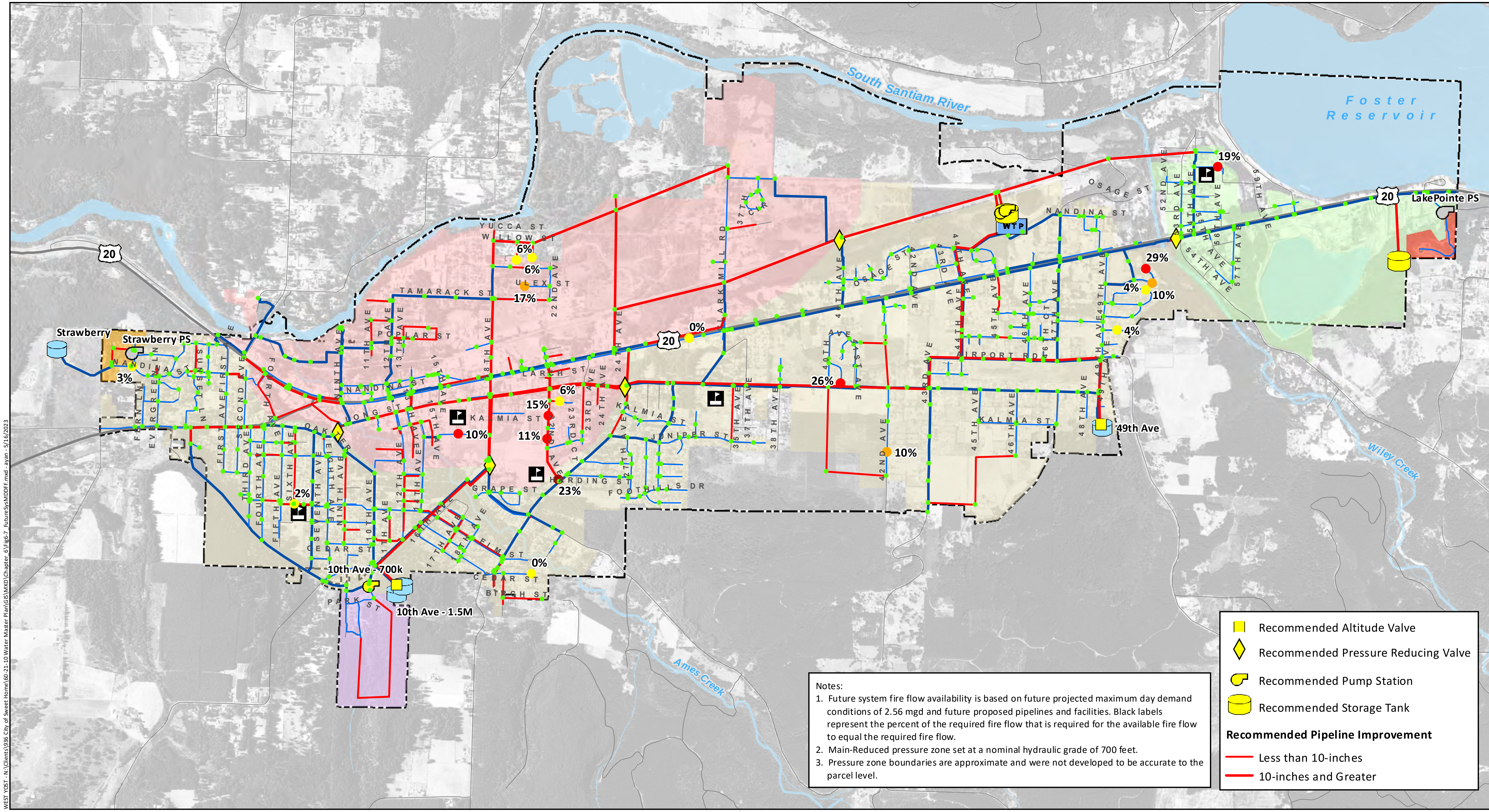
6.3 SUMMARY OF RECOMMENDED IMPROVEMENTS

The recommended improvements proposed to eliminate the water system capacity and performance deficiencies identified in the preceding evaluations are summarized below. These recommendations only identify improvements at a master planning level and do not constitute a design of such improvements. Subsequent detailed design will be required to determine the exact sizes and/or locations of these proposed improvements. The estimated costs for these recommended improvements are discussed in *Chapter 9 Capital Improvements Program*.

Figure 6-8 summarizes all improvements recommended for the City's water system, by diameter, to meet the City's performance criteria. Improvements shown in Figure 6-8 can be categorized as follows:

- **Small-Diameter Mains Improvements:** Replacement of all City-owned pipelines 2-inches in diameter. All pipelines are assumed to be replaced with 8-inch for looped pipelines and 6-inch for dead-end pipelines. This is included in the CIP as two line items.
- **Capacity or Reliability Improvements:** Proposed improvements to meet the performance criteria described in Chapter 4 and long-term operational goals identified by the City (refer to Section 6.2.1). These improvements include the replacement of existing pipelines and the construction of new pipelines, pump stations, reservoirs, and PRVs. These improvements are included in the CIP as individual projects.
- **Fire Flow Improvements:** Proposed improvements to meet fire flow performance criteria described in Chapter 4. These improvements include the replacement of existing pipelines and the construction of new pipelines. These improvements are included in the CIP as individual projects.

Detailed discussion and depiction of each recommended improvement by improvement type and individual project is included in *Chapter 9 Capital Improvement Program*.



Junction Meets Fire Flow Requirement

Flow to Meet Fire Flow Requirement at Deficient Junctions

- Less than 200 gpm
- 200 - 500 gpm
- 500 - 1,000 gpm
- 1,000 - 2,000 gpm

Recommended Pressure Zones

- Foster
- Main
- Main-Reduced
- 10th Ave
- Strawberry
- LakePointe

WTP Existing Water Treatment Plant

Existing Storage Tank

Potable Water Pump Station

Existing System Pipelines

- Diameter Less than 10-inches
- Diameter 10-inches and Greater

City Limit

School

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N

0 900 1,800

Scale in Feet

WEST YOST

Figure 6-7
Future System Recommended Improvements
Future MDD - Fire Flow Availability

City of Sweet Home
Water Master Plan

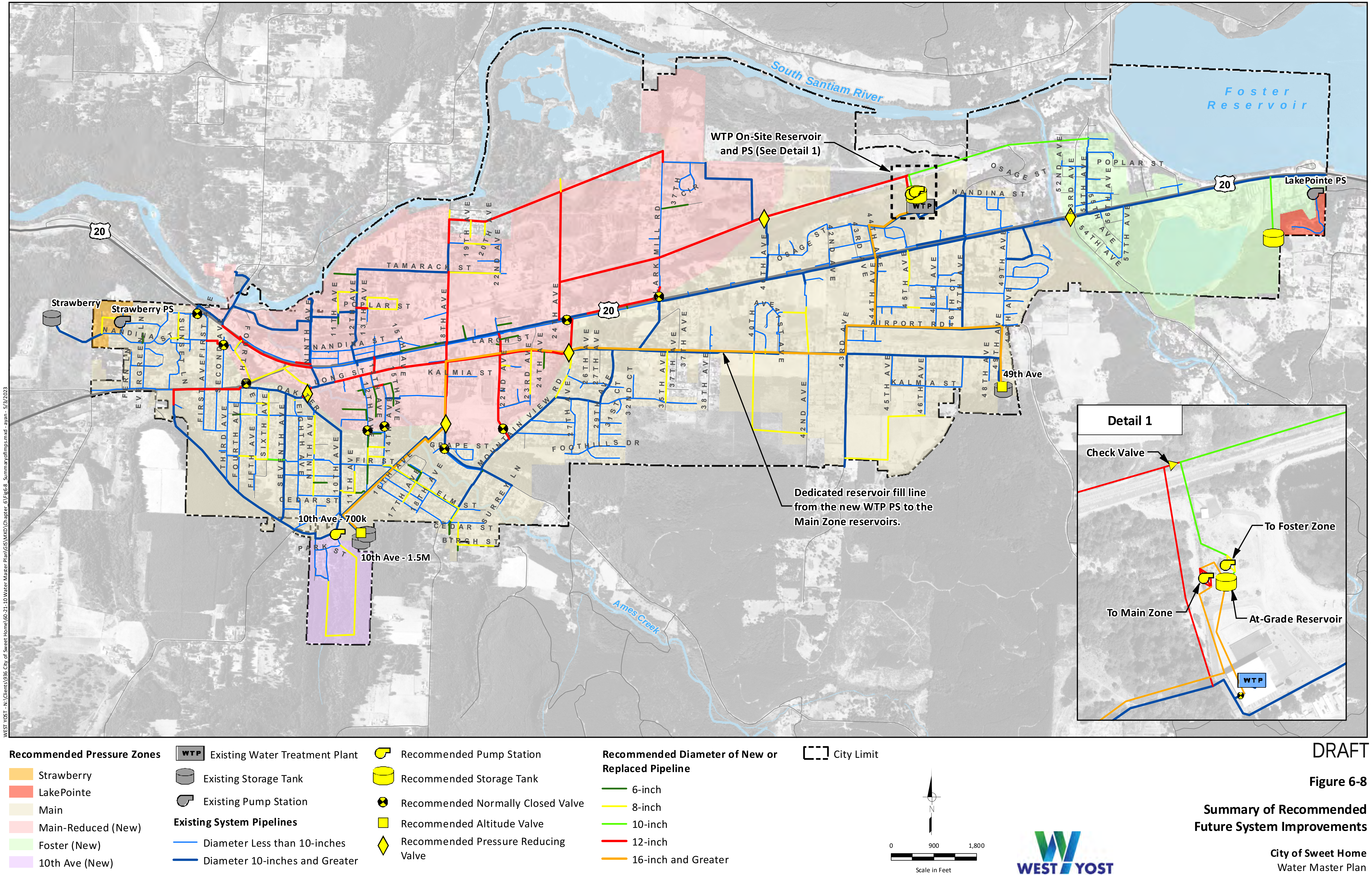

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Figure 6-8

Summary of Recommended Future System Improvements

City of Sweet Home
Water Master Plan

CHAPTER 7
Water Treatment Plant Evaluation and Upgrades

This chapter presents an evaluation of the City's existing WTP and identifies needs for meeting water service requirements and performance standards over the 20-year Master Plan horizon. The analysis includes both system capacity and performance evaluations based on the performance criteria presented in *Chapter 4 Design and Performance Criteria*. The system capacity evaluation includes an evaluation of existing supply, treatment, and storage capacity under existing and projected water demands.

The following sections present the evaluation methodology and results for the Water Treatment Plant:

- Water Treatment Plant Overview
- Recommended Improvements
- WTP Annual O&M Projects

7.1 WATER TREATMENT PLANT OVERVIEW

The raw water intake for the water treatment plant was constructed in 2006. It begins at the Foster Dam where the City diverts raw water from the Foster Reservoir through a fish/debris screen. Raw water then flows through an above ground 24-inch DI pipe for approximately 600 feet before transitioning to below grade through a 30-inch HDPE pipe. The pipe runs for approximately 4,600 feet and discharges into a raw water wet well north of the City's WTP. From the raw water wet well, flows are pumped to the WTP. More information on Foster Dam and the raw water intake can be found in Chapter 2.

The City's WTP was constructed in 2009 and includes three (3) treatment trains that each include a raw water pump, a chemical feed system, static mixers, a tube clarifier, adsorption clarifier media, mixed media filter and chemical disinfection. The treated and disinfected water then progresses through a 10-mgd baffled clearwell, where three (3) finish water (FW) pumps deliver the finished water to the City's water distribution system. The treatment facility also includes two backwash ponds north of the treatment building. The City's water treatment plant site location and facility components are shown in Figure 7-1 and Figure 7-2, respectively.

7.1.1 System Capacity Analysis

7.1.1.1 Water Treatment Capacity

The nominal capacity of each parallel train system is 1,400 gpm, for a total WTP capacity of 4,200 gpm, or approximately 6.0 mgd. Assuming there is a fully redundant filter, the firm WTP capacity is 2,800 gpm, or approximately 4.0 mgd. See Chapter 2 for more information about the water treatment facility capacity.

7.1.1.2 Projected Water Production Evaluation

As described in *Chapter 3 Water Demand,* the existing average day demand is 0.64 mgd, based on historical annual water consumption, with an associated average day production of 0.85 mgd. The City's 20-year projected average day water production of 1.1 mgd. The recommended peaking factor for maximum day demand is 2.4 times average day demand. Therefore the current maximum day production requirement to meet maximum day demand is 2.0 mgd and the 20-year projected water production requirement is estimated at 2.6 mgd.

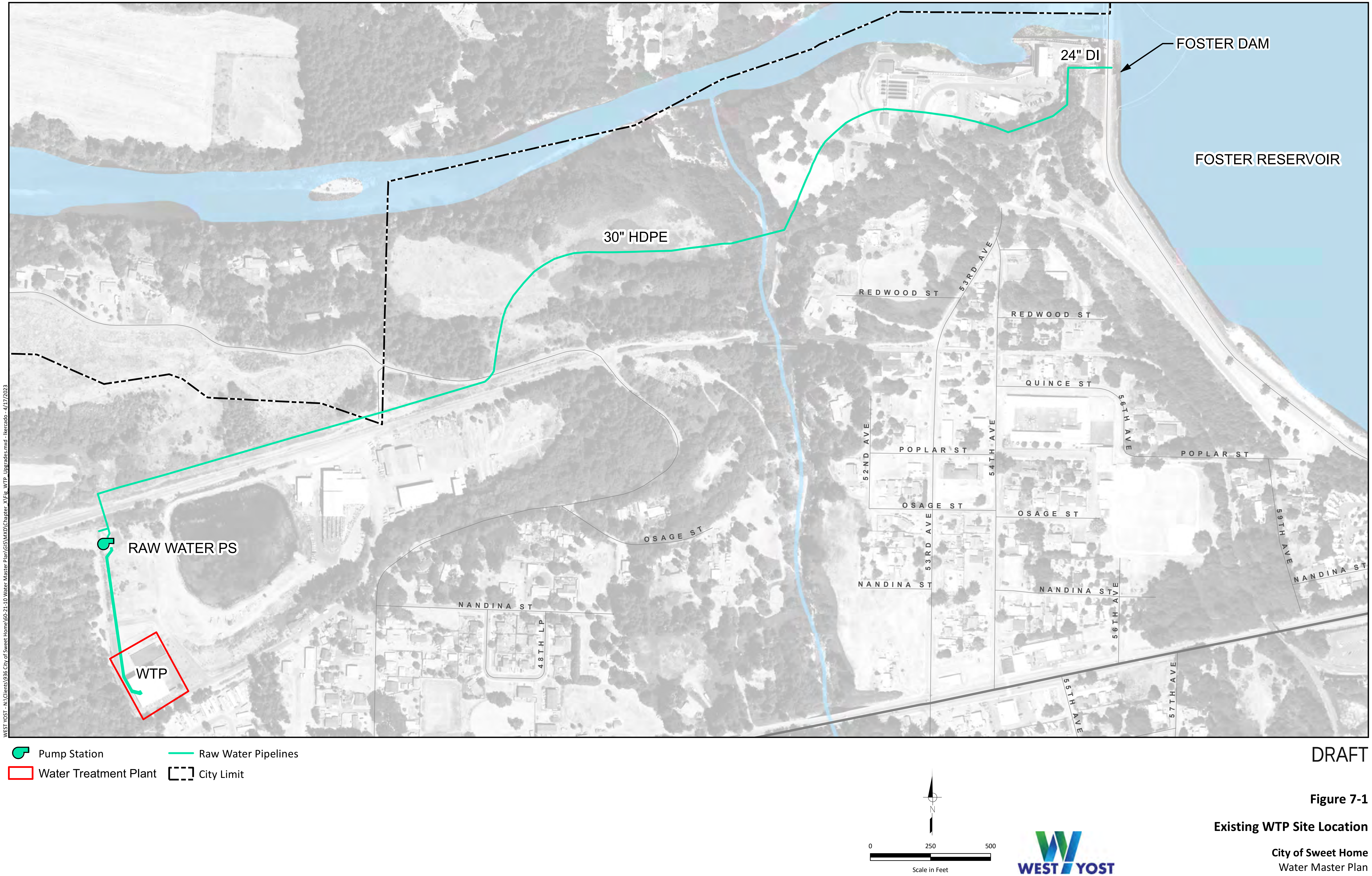

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Figure 7-1

Existing WTP Site Location

City of Sweet Home
Water Master Plan

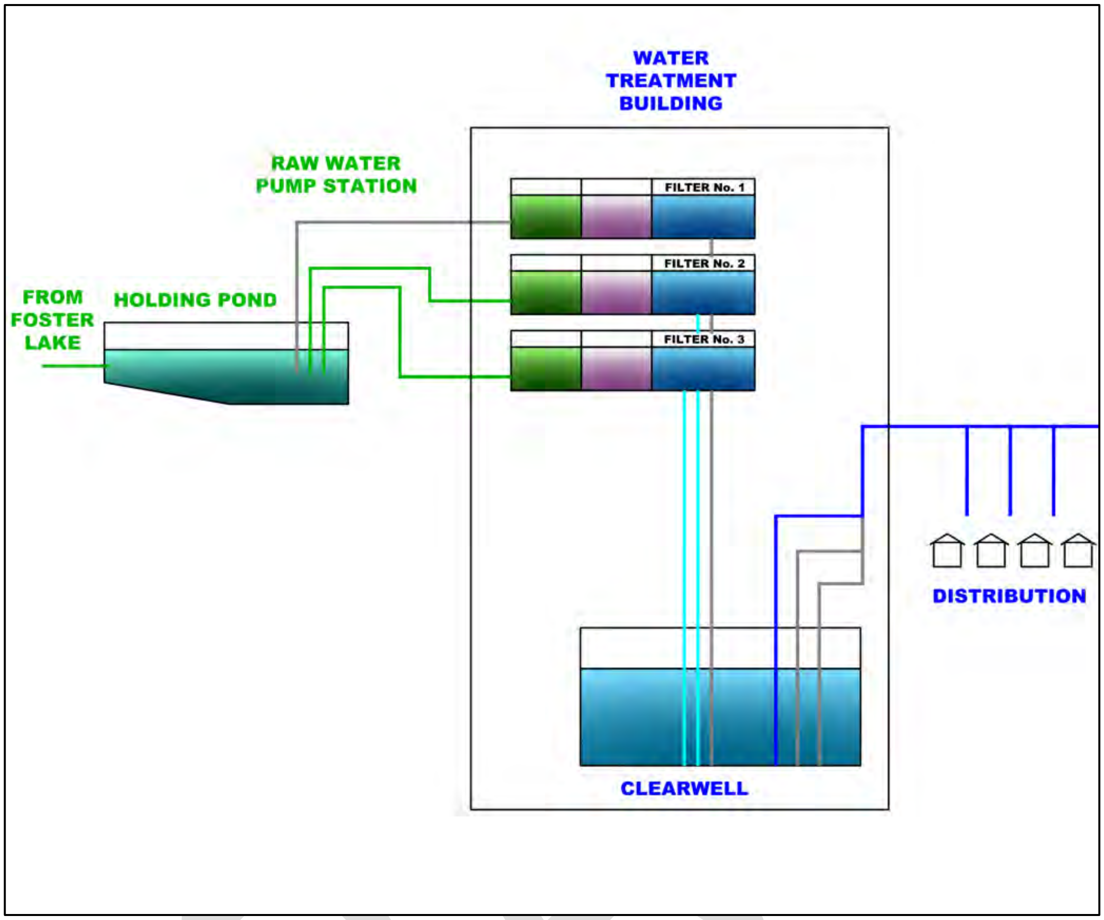

Figure 7-2. Water Treatment Plant Facility Diagram

7.1.1.3 Overall WTP Capacity Evaluation

The firm capacity of the water treatment plant is approximately 4.0 mgd compared with current and projected required maximum day production of 2.0 mgd and 2.6 mgd, respectively. Therefore, the existing WTP has more than sufficient capacity to meet current and future demands over the 20-year Master Plan horizon.

7.1.2 Recent Upgrades

The City is currently finishing a project to add variable frequency drives (VFDs) to the three existing FW pumps and a new backwash pump (BP) to alleviate distribution system pressure issues. At the time of this WMP, the City is currently awaiting delivery of a new BP that is being installed in the location of a future FW pump which the City does not anticipate needing over the 20-year Master Plan horizon. Figure 7-3 shows the FW and new BW pumps at the WTP.

The new BW pump will pull directly from the clearwell for backwashing. The current BP pulls water from the City's distribution system which creates severe pressure fluctuation through the system. The addition of the new BP and water source will eliminate this issue. The old backwash system will be kept in place as backup backwash water supply with the addition of a new 14-inch PRV on the BP discharge piping.

In early 2023, electrical upgrades were completed to accommodate the new loads from the VFDs and BP upgrades. The electrical upgrades for the new BP include a new MCC section with soft start, replacement of the existing power conductors, replacement of the circuit breaker trip plug. Additionally, a new control panel was included for the FW pumps.

7.2 RECOMMENDED IMPROVEMENTS

West Yost conducted a condition assessment of the WTP with City staff to identify any potential deficiencies in the treatment process. Even though the WTP has sufficient capacity for the next 20-year period, some improvements were identified. Below is a list of recommended improvements at the WTP:

7.2.1 WTP Project #1 – Filter Feed Piping Manifold System

This proposed project will upgrade the raw water feed pipelines entering each filter to connect them together in a manifold system with actuated valves to allow any filter to be operated with any raw water pump. This will improve reliability and redundancy of the existing filters and raw water pump station. The upgrades are shown in Figure 7-4.

The estimated cost of the manifold system is $77,000 as summarized in Table 7-1 below.

Table 7-1. Preliminary Costs for Filter Feed Piping Manifold System

Description	Total, dollars
Valves	22,000
Tee	15,000
Ductile Iron Pipe	10,000
General Conditions (12%)	2,000
Contractor Overhead (15%)	7,000
Engineering and Design (20%)	9,000
Contingency (25%)	12,000
Total	**$77,000**

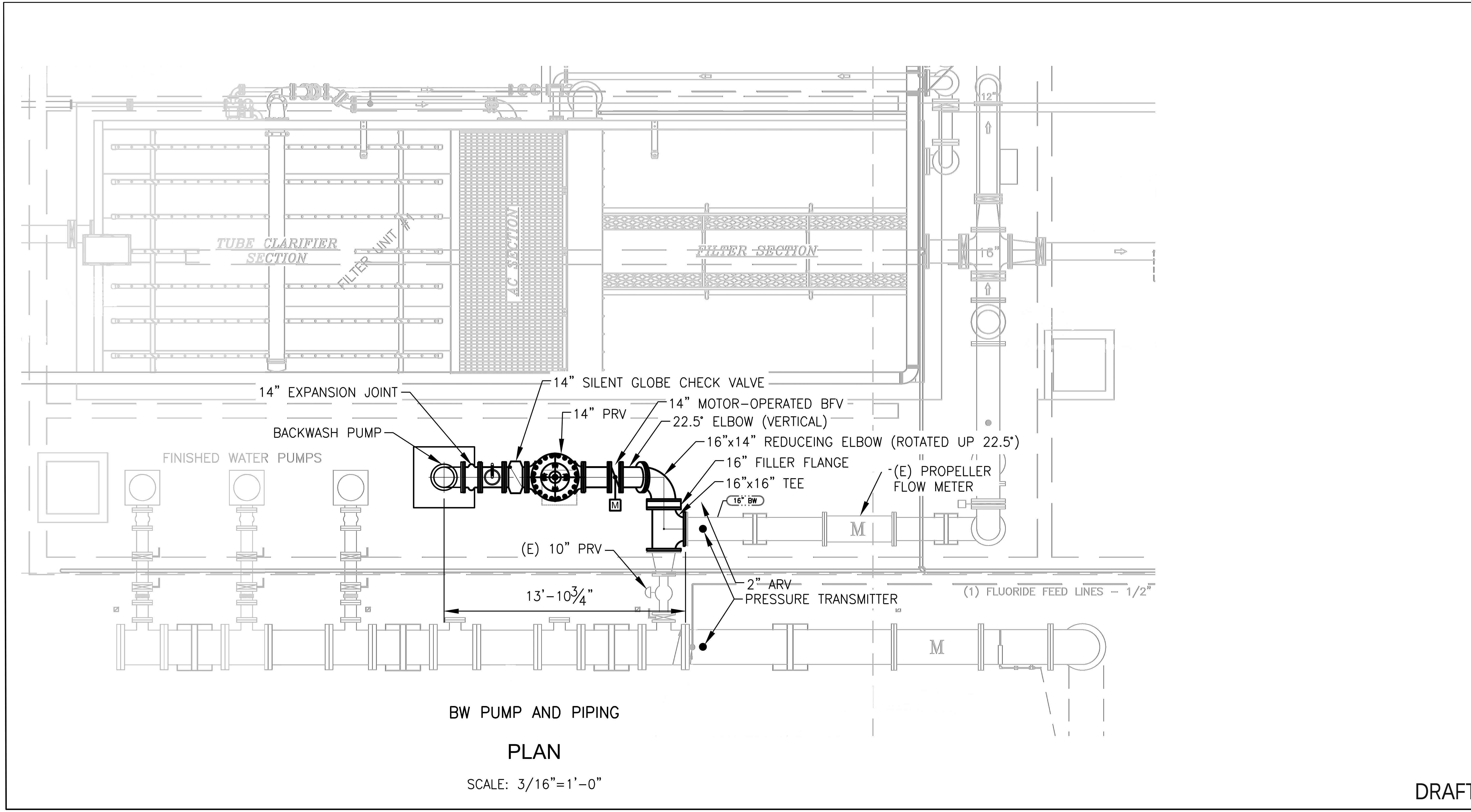

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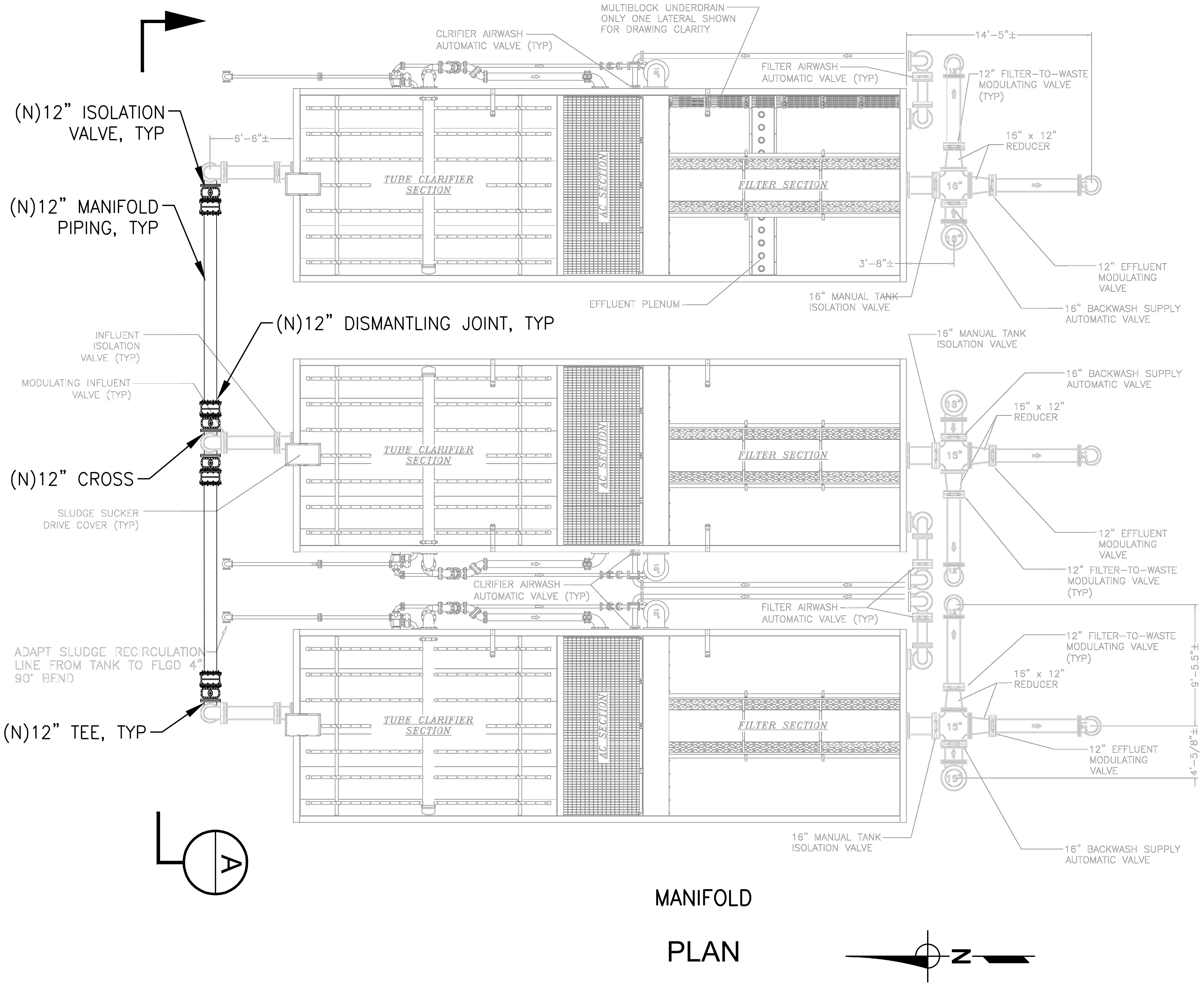
Figure 7-3

Backwash Pump Upgrades

City of Sweet Home
Water Master Plan



MANIFOLD

PLAN

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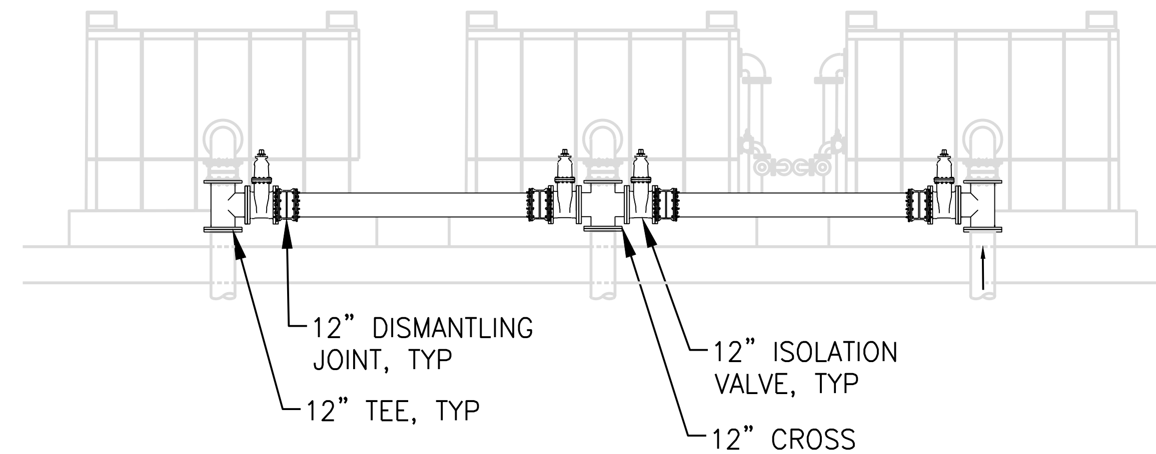

SECTION A-A

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Figure 7-4

Manifold Upgrade

City of Sweet Home
Water Master Plan

7.2.2 WTP Project #2 - New WTP Standby Generator and ATS

To improve reliability of the WTP to produce water during periods of extended power outages, it is recommended that a new diesel engine standby generator and automatic transfer switch (ATS) be installed at the plant. The estimated cost of the new standby generator and ATS is $984,000 as summarized in Table 7-2 below.

Table 7-2. Preliminary Costs for Standby Generator

Description	Total, dollars
Switch Gear & ATS	350,000
Standby Generator	250,000
General Conditions (12%)	24,000
Contractor Overhead (15%)	90,000
Engineering and Design (20%)	120,000
Contingency (25%)	150,000
Total	**$984,000**

7.2.3 WTP Project #3 – Filter Sludge Removal System Replacement

This proposed project involves replacement of the sludge removal systems in each of the existing WTP filters to improve WTP performance. The system will be similar to the vacuum system shown in Figure 7-5 below.

Figure 7-5. Meurer Research Hoseless Vacuum Sludge Collector

The estimated cost for replacement of each filter sludge removal system is $250,000 and the total estimated cost for all 3 filters is $750,000.

7.2.4 WTP Project #4 – New Sludge Drying Bed

A new sludge drying bed is needed at the WTP to improve the ability to dry solids from the sludge removal systems and keep the WTP in operation. A proposed location for the sludge drying bed expansion is just north of the WTP building on the other side of the access road.

The estimated cost for the new sludge drying bed is $33,000 as summarized in Table 7-3 below.

Table 7-3. Preliminary Costs for Sludge Drying Beds

Description	Total, dollars
Excavation	6,000
Concrete	13,000
Sand and Gravel Backfill	1,000
General Conditions (12%)	1,000
Contractor Overhead (15%)	3,000
Engineering and Design (20%)	4,000
Contingency (25%)	5,000
Total	**$33,000**

7.2.5 WTP Recommended Projects Summary

Table 7-4 below summarizes the recommended WTP projects. It is recommended that these projects be completed over the next 5 years.

Preliminary costs for each upgrade were developed and are shown in the Table 7-4 below.

Table 7-4. Preliminary Costs for Recommended Improvements

Improvement	Cost, dollars[a]
WTP Project #1: Filter Feed Maniform Piping Upgrades	77,000
WTP Project #2: New Standby Generator and ATS	984,000
WTP Project #3: Filter Sludge Removal System Replacement	750,000
WTP Project #4: New Sludge Drying Bed	33,000
Total	**$1,844,000**

(a) Includes contractor overhead and profit, engineering design and contingency.

7.3 OPERATION AND MAINTENANCE PROJECTS

In addition to near-term WTP projects identified in Section 6.2, the City also frequently needs to complete O&M upgrades at the WTP. These upgrades are difficult to plan for or schedule because they can come up quickly when equipment breaks down. In addition, some specific issues have been identified by the City staff during normal daily operations. These items require more rigorous upgrades and need to be planned.

City staff maintain a list of potential O&M projects that can be completed if time and opportunity arise. These include:

- Upgrade the fluoride system (currently in progress).
- Upgrade SCADA (currently in progress).
- Upgrade CL2 pump to work remotely from setpoints in SCADA. The pumps are currently being manually adjusted.
- Automate soda ash system and install inline pH meters on each raw water line downstream of the soda ash injection point so that the soda ash can run from setpoints in SCADA.
- Upgrade pre and post polymer chemical pumps to run on setpoints from SCADA.
- Replace the roof.
- Modify controls and pumping to allow raw water pumps to pump into a common header where chemicals are added which then feeds the individual trains.

Rather than estimate these small O&M projects individually and program them along with the CIP, it is recommended that the City create a WTP Annual O&M Projects line item in the annual budget for these projects. An annual budget of $75,000 is recommended as a starting point, but the costs for these projects should be monitored and the annual budget updated if/as needed.

CHAPTER 8
Seismic Risk and Mitigation Plan

This chapter summarizes the seismic resiliency of the City's water system. This resiliency effort evaluates the seismic hazards present within the City's water service area with their potential impacts to the water system after a major seismic event, and then recommends mitigation approaches.

The following sections describe the key components of this chapter:

- Introduction with background information
- Water System Backbone with identification of essential water facilities, and critical customers
- Seismic Resiliency Evaluation including a geotechnical and structural assessments, and pipe fragility
- Seismic Resiliency Evaluation Results
- Mitigation of Seismic Hazards

8.1 INTRODUCTION

The Pacific Northwest is located near an active tectonic plate boundary, the Cascadia Subduction Zone (CSZ), a zone prone to generate large earthquakes. A magnitude 9.0 Cascadia seismic event in this zone would pose a significant enough risk to the communities and the economy that an Oregon Resilience Plan (ORP) was developed in 2013. This plan outlines steps that can be taken over a 50-year period to reach desired resilience targets and recovery goals; this includes upgrades, retrofits, or rebuilding over the 50-year timeframe of key water supply, treatment, and distribution elements to withstand a Cascadia subduction zone earthquake. The City is following these recommendations for its water system. Figure 8-1 presents the 2013 ORP's target states of recovery for domestic water supply in the Willamette Valley region (Valley) which applies to the City's service area and compares it to the expected performance if the earthquake were to have occurred at the time the 2013 ORP was written.

As shown in Figure 8-1, the timeframes for recovery for existing water systems (Current State) are generally not able to meet the target recovery goals. These gaps in time difference illustrate that seismic improvements are needed to achieve the performance goals. Capital investment would be necessary to improve water infrastructure resiliency and enhance public policy over the years. The resilience of the City's water system will be integral to emergency needs and recovery.

The 2013 ORP also included the development of earthquake scenario maps produced by the Oregon Department of Geology and Mineral Industries (DOGAMI). These maps show the results of simulated strong shaking, impacted zones, estimated inundation areas, estimated amount of ground failure and movement that are all likely to occur during a magnitude 9.0 earthquake in the region.

KEY TO THE TABLE

TARGET TIMEFRAME FOR RECOVERY:

Desired time to restore component to 80–90% operational	G
Desired time to restore component to 50–60% operational	Y
Desired time to restore component to 20–30% operational	R
Current state (90% operational)	X

TARGET STATES OF RECOVERY: WATER & WASTEWATER SECTOR (VALLEY)

	Event occurs	0–24 hours	1–3 days	3–7 days	1–2 weeks	2 weeks–1 month	1–3 months	3–6 months	6 months –1 year	1–3 years	3 + years
Domestic Water Supply											
Potable water available at supply source (WTP, wells, impoundment)		R	Y		G			X			
Main transmission facilities, pipes, pump stations, and reservoirs (backbone) operational		G					X				
Water supply to critical facilities available		Y	G				X				
Water for fire suppression—at key supply points		G		X							
Water for fire suppression—at fire hydrants				R	Y	G			X		
Water available at community distribution centers/points			Y	G	X						
Distribution system operational			R	Y	G				X		

Figure 8-1. 2013 ORP's Target States of Recovery for Domestic Water Supply in the Willamette Valley Region[1]

[1] Oregon Seismic Safety Policy Advisory Commission (OSSPAC). February 2013. *Oregon Resilience Plan.* Figure 8.19: Water & Wastewater Sector: Valley Zone.

According to the Map of Earthquake and Tsunami Damage Potential developed for the 2013 ORP[2], the City is located in a Zone ranging from VI to VIII, equivalent to an area from light to moderate/heavy Damage Potential following a magnitude 9.0 CSZ earthquake. Due to its potential risk, a seismic risk assessment and mitigation plan for the City's water system shall be developed in accordance with the OHA requirements and the 2013 ORP goals.

OAR 333-061-0060 (J)

(J) A seismic risk assessment and mitigation plan for water systems fully or partially located in areas identified as VII to X, inclusive, for moderate to very heavy damage potential using the Map of Earthquake and Tsunami Damage Potential for a Simulated Magnitude 9 Cascadia Earthquake, Open File Report 0-13-06, Plate 7 published by the State of Oregon, Department of Geology and Mineral Industries.

i. *The seismic risk assessment must identify critical facilities capable of supplying key community needs, including fire suppression, health and emergency response and community drinking water supply points.*
ii. *The seismic risk assessment must identify and evaluate the likelihood and consequences of seismic failures for each critical facility.*
iii. *The mitigation plan may encompass a 50-year planning horizon and include recommendations to minimize water loss from each critical facility, capital improvements or recommendations for further study or analysis*

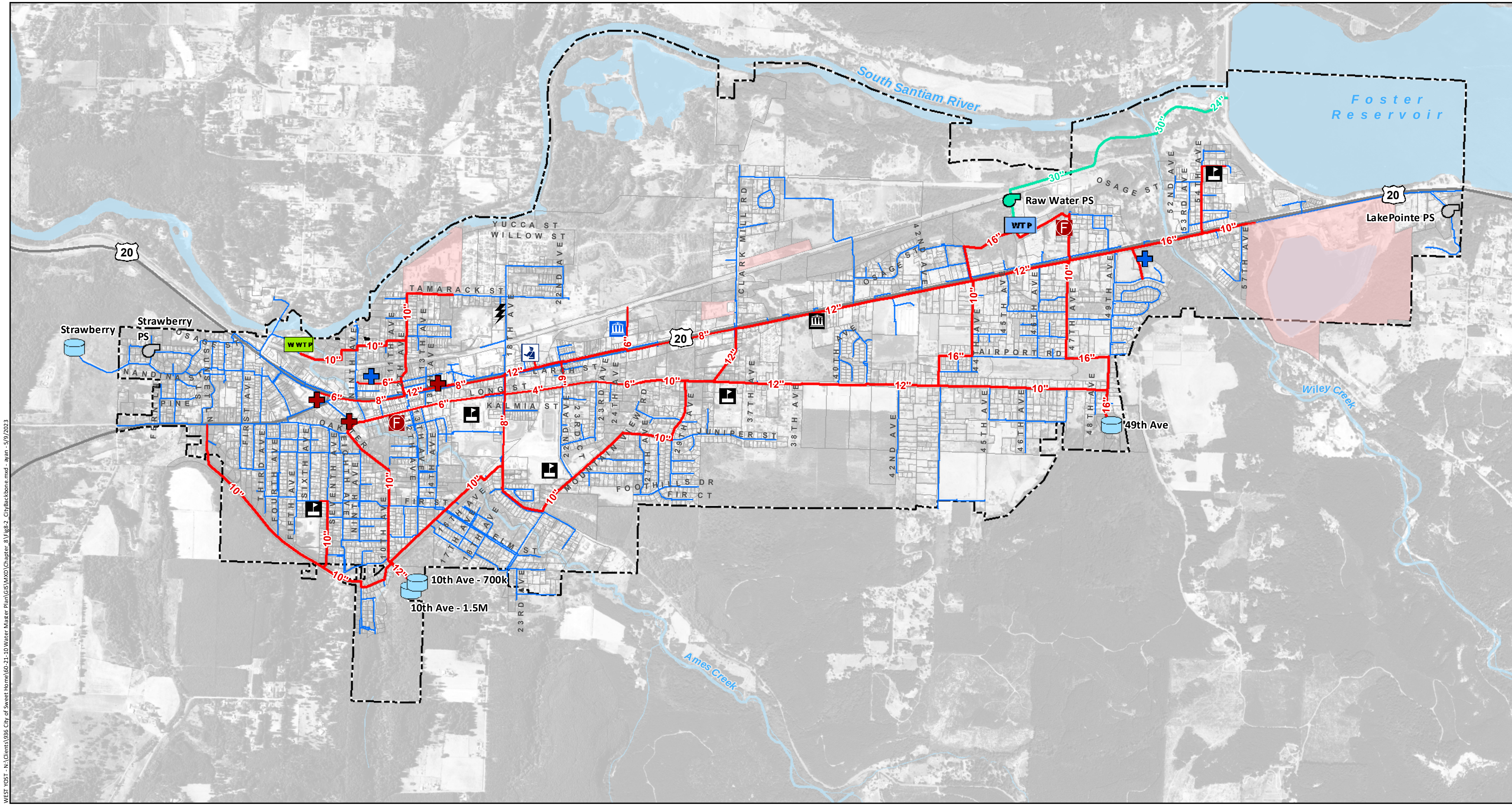
The objectives of this resilience assessment are to ensure reasonable levels of service for drinking water supplies and to help planning the improvement of the resiliency of the City's critical water system backbone.

8.2 WATER SYSTEM BACKBONE

A water system backbone is the infrastructure required to maintain adequate supply to essential facilities and critical customers in the City for post-earthquake response, public health and safety. Using the 2013 ORP guidelines, backbone infrastructure and water facilities were identified for the City's water system including the raw water intake and pump station, the WTP, the City's reservoirs and associated pump stations, and the critical pipelines. A map of the backbone system for the City is shown on Figure 8-2.

Following a seismic event, water supply will be disrupted and many of the residential, commercial, and industrial water services will be damaged. It is important to identify critical water customers for whom water service shall be uninterrupted or quickly restored. This list consists of City Hall, police departments, fire stations, the Public Works building, healthcare facilities, schools, and other utilities (see Figure 8-2 for locations). The water system backbone identifies transmission and distribution mains that supply and connect the critical customers and key water facilities. The key facilities and their connection points are shown on Figure 8-2.

[2] Madin, I.P. & Burns, W.J. 2013. *Map of Earthquake and Tsunami Damage Potential for a Simulated Magnitude 9 Cascadia Earthquake.* Assessed at https://digital.osl.state.or.us/islandora/object/osl%3A55566/datastream/OBJ/view.



- Potable Water Pump Station
- Raw Water Pump Station
- WTP Water Treatment Plant
- Storage Tank
- Raw Water Pipeline
- Non-Backbone Pipeline
- Backbone Pipeline
- Industrial Customers
- City Limit

Critical Utilities

- WWTP Wastewater Treatment Plant
- Sweet Home Public Works
- Power Station

Critical Locations

- Fire Station
- City Hall
- Police Station
- Healthcare Facility
- Assisted Living
- School

Notes:
1. The finished water pump station is located on-site at the City's water treatment plant.
2. The 0.3 MG 10th Ave tank constructed in 1938 is currently offline and is not pictured.

DRAFT

N

0 900 1,800
Scale in Feet

WEST YOST

Figure 8-2

Backbone Identification Map

City of Sweet Home
Water Master Plan

8.3 SEISMIC RESILIENCY EVALUATION

To help the City prepare and appropriately invest in resilience planning for its water system backbone, geotechnical and structural seismic hazards assessments were developed. A 9.0 CSZ earthquake was selected for the earthquake hazards analysis, consistent with the 2013 ORP. The maximum considered earthquake (MCE_R) was not considered due to the long length of its estimated 2,475-year recurrence interval.

This section includes the methodology used to evaluate the seismic hazards and pipeline fragility within the City's water backbone system.

8.3.1 Geotechnical Seismic Risks and Hazards Mapping

McMillen Jacobs Associates was contracted to complete a geotechnical seismic hazards evaluation of the City's service area. The first step was to identify the geologic setting under the City, then to analyze and delineate the peak ground velocity (PGV) and permanent ground deformations (PGD) to be expected from a magnitude 9.0 CSZ earthquake.

The City water service area is located in the foothills of the Western Cascades which were formed by a series of volcanic events 35 to 17 million years ago. The structural basement of this region is the Paleogene, composed of non-marine volcaniclastic sedimentary rocks, tuff, basaltic andesite, andesite, and dacite. This Paleogene layer is overlain by basalt lavas, tuff, and sedimentary rocks, followed by a top layer of sediments consisting of alluvium, colluvium, landslide deposits, and unconsolidated gravel and sand, with lenses of silt and clay.

Analysis of the seismic hazards in the City's service area is based on geological information, geotechnical explorations, historic well logs, background data, and available earthquake scenario maps (DOGAMI maps). Seismic hazards to be estimated include strong ground shaking (peak ground velocity and acceleration), liquefaction settlement, lateral spreading displacement, and seismic-induced landslides. Spectral accelerations were estimated for a CSZ earthquake. Although a MCE_R was not considered for the earthquake hazards analysis as mentioned in Section 7.3, McMillen Jacobs Associates also included spectral accelerations for a MCE_R.

Following these findings, McMillen Jacobs developed maps illustrating these hazards in relation to the City's backbone system. The complete seismic hazards evaluation and mapping technical memorandum is included in Appendix B.

8.3.2 Structural Seismic Resiliency Evaluation

ACE was contracted to complete a structural seismic evaluation of the existing critical water structures in the water treatment and distribution system of the City. The primary purpose of this evaluation is to identify the potential structural and seismic deficiencies of each critical structure. This evaluation is based on review of available record drawings, geotechnical seismic hazards evaluation data provided by McMillen Jacobs Associates, and a site observation of each structure. The Tier 1 level of ASCE 41-17 "Seismic Evaluation and Upgrade of Existing Buildings" was used for the evaluation with a performance level of "Immediate Occupancy". Structural and non-structural items were assessed and compared to current prescribed loading and detailing requirements for lateral (wind/seismic) loading. Non-structural items include utilities, fixtures, equipment, finishes and furnishings. The detailed and complete structural evaluation is provided in a technical memorandum in Appendix C.

8.3.3 Pipeline Fragility Evaluation

To estimate the likelihood of damage to buried pipes in a seismic event, the American Lifelines Alliance (ALA) developed methods published in the report *Seismic Fragility Formulations for Water Systems* (ALA 2001) for estimating seismic fragility for water pipes. These methods are based on the frequency of pipe breaks in past earthquakes and correlating this with the ground shaking and measured ground movements (from liquefaction and landslides) at the site of the break. A break is defined as pipe damage severe enough to require a repair. Water agencies frequently use these methods to estimate the seismic resiliency of their water system backbone pipes.

The ALA guideline recommends using two pipe vulnerability functions as shown in Table 8-1 to evaluate the repair rates (RR) for a large inventory of pipelines such as a water distribution system. The first function estimates a RR per 1,000 LF of pipe due to seismic wave propagation (ground shaking), and the second function estimates a RR per 1,000 LF of pipe due to permanent ground deformation (liquefaction, lateral spreading, and seismic landslides).

Table 8-1. Buried Pipe Vulnerability Functions

Hazard	Vulnerability Function	Lognormal Standard Deviation, β
Wave Propagation	RR=K1 x 0.00187 x PGV	1.15
Permanent Ground Deformation	RR=K2 x 1.06 x PGD0.319	0.74

RR = repairs per 1,000 LF of pipe
PGV = peak ground velocity (in/sec)
PGD = permanent ground deformation (in)

In Table 8-1, K1 and K2 are empirical fragility factors to scale the repair rates for different pipe diameters, pipe materials, and joint types, which can either increase or decrease the base pipe break rate. K1 represents the strength and flexibility of the pipe material to withstand ground shaking. K2 represents the strength and flexibility of the pipe joint to resist separation during ground deformation.

The results of these repair rate values can then be evaluated to assess the vulnerability or fragility of the backbone pipelines to seismic damage.

8.4 SEISMIC RESILIENCY EVALUATION RESULTS

As shown in Figure 8-2, the City's critical water facilities include the raw water intake and pump station, the water treatment plant, the LakePointe Pump Station, the Strawberry Reservoir, pump station and vault, the 10th Avenue Reservoirs, and the 49th Avenue Reservoir.

The results of the geotechnical and structure analyses indicate that the majority of the City's service area is not located within a seismic hazard zone and most of the critical water facilities are in reasonable structural condition. The ground shaking hazard is moderate, and the liquefaction and lateral spreading hazards are low. Landslide hazard is low as well due to the relative flatness of the City, except along the southern boundary of the service area where steeper slopes are present. Landslide hazard may impact the 10th Avenue and 49th Avenue Reservoirs which are located near steep slopes.

The results of the seismic resiliency evaluation for the critical water facilities are summarized below. Additional details regarding the analyses of these facilities are provided in Appendices B and C.

8.4.1 Raw Water Intake and Pump Station

8.4.1.1 Raw Water Intake

The Raw Water Intake is located on the Foster Reservoir Dam. The intake structure was built in 2007 and consists of a slab on grade with CMU (Concrete Masonry Unit) block walls supporting a wood frame roof. Table 8-2 summarizes the findings and recommendations for improvements.

Table 8-2. Raw Water Intake – Seismic Evaluation Summary

Potential	Description
Seismic	• 5-10 in/sec ground shaking intensity (PGV); low risk of liquefaction, lateral spreading, and seismic landslides.
Structural	• No deficiencies were found.
Non-Structural	• Lack of rain gutter on the back of the roof contributing to some minor exposure or scour on the downhill side of the building.

8.4.1.2 Raw Water Pump Station

The Raw Water Pump Station is located north of the WTP and was built in 2008. The pump station consists of a CMU block pump house with an on-grade slab supporting a wood frame roof, and an underground concrete wet well with a maximum depth of 10 feet. Table 8-3 summarizes the findings and recommendations for improvements.

Table 8-3. Raw Water Pump Station – Seismic Evaluation Summary

Potential	Description
Seismic	• 5-10 in/sec ground shaking intensity (PGV); low risk of liquefaction, lateral spreading, and seismic landslides.
Structural	• No deficiencies were found.
Non-Structural	• None.

8.4.2 Water Treatment Plant

8.4.2.1 Water Treatment Building

The Water Treatment Building was built in 2008 surrounded by a concrete retaining wall on the south side and CMU blocks along the other perimeter sides. The main floor of the building consists of a slab on grade with a below grade concrete clearwell on the east side. The building is framed by Pre-Engineered Metal Building steel frames with light gauge metal roof purlins. The west portion of the building contains a wood framed mezzanine for staff offices, IT room, a laboratory, and a meeting room. Table 8-4 summarizes the findings and recommendations for improvements.

Table 8-4. Water Treatment Building – Seismic Evaluation Summary	
Potential	Description
Seismic	• 5-10 in/sec ground shaking intensity (PGV); low risk of liquefaction, lateral spreading, and seismic landslides.
Structural	• The mezzanine is open to the east toward the filters making it a 3-sided diaphragm. No Shear walls are provided for lateral resistance of the mezzanine diaphragm along the east side. • The height to thickness ratio of the masonry walls exceed the recommended limits. • The stair opening in the mezzanine diaphragm is adjacent to the exterior masonry wall and exceeds the recommended limits. • The stair opening in the mezzanine diaphragm is considered a plan irregularity. There is a lack of tensile capacity around the stair opening in the mezzanine diaphragm. • The mezzanine diaphragm was not noted to have blocking at the plywood panel edges. The unblocked diaphragm exceeds allowable limits and aspect ratios when subject to east-west lateral loading.
Non-Structural	• Several items are suspended from the structure and are free to swing or move but may damage themselves or adjoining components. • There are several pieces of equipment more than 6 feet tall that should be anchored to the floor or adjacent walls. • Conduit greater than 2.5 inches should have flexible couplings. • The condensation buildup above the insulation should be addressed to prevent further failure of the insulation. • The rust and corrosion around the base of the steel columns should be treated, repaired, and properly coated to prevent further deterioration.

8.4.2.2 Water Treatment Pond

The Water Treatment Pond was built in 2008 at the same time as the Water Treatment Building and located just north of the building. The backwash pond consists of two adjacent concrete structures. The divider wall is made of a reinforced concrete with a weir. Table 8-5 summarizes the findings and recommendations for improvements.

Table 8-5. Water Treatment Pond – Seismic Evaluation Summary	
Potential	Description
Seismic	• 5-10 in/sec ground shaking intensity (PGV); low risk of liquefaction, lateral spreading, and seismic landslides.
Structural	• No deficiencies were found.
Non-Structural	• None.

8.4.3 LakePointe Pump Station

The LakePointe Pump Station is located on the east side of the City just off of Highway 20 near Foster Reservoir. The pump station structure was built in 2016 and consists of a slab on grade with CMU block walls supporting a wood framed roof trusses. Table 8-6 summarizes the findings and recommendations for improvements.

Table 8-6. Lake Pointe Pump Station – Seismic Evaluation Summary

Potential	Description
Seismic	• 5-10 in/sec ground shaking intensity (PGV); low risk of liquefaction, lateral spreading, and seismic landslides.
Structural	• No deficiencies were found.
Non-Structural	• None.

8.4.4 Strawberry Reservoir and Pump Station

8.4.4.1 Strawberry Reservoir

The Strawberry Reservoir was built in 2001 at a location near the western limit of the City. The reservoir is a bolted steel tank on a concrete foundation on grade with a capacity of 110,000 gallons. Table 8-7 summarizes the findings and recommendations for improvements.

Table 8-7. Strawberry Reservoir – Seismic Evaluation Summary

Potential	Description
Seismic	• 5-10 in/sec ground shaking intensity (PGV); low risk of liquefaction, lateral spreading, and seismic landslides.
Structural	• No deficiencies were found but the nuts of the existing anchor bolts should be tightened.
Non-Structural	• None.

8.4.4.2 Strawberry Vault

The Strawberry Vault is located at the reservoir site and built at the same time as the tank. The vault structure consists of a slab on grade with CMU block walls supporting a grating floor and a wood framed roof. Table 8-8 summarizes the findings and recommendations for improvements.

Table 8-8. Strawberry Vault – Seismic Evaluation Summary	
Potential	Description
Seismic	• 5-10 in/sec ground shaking intensity (PGV); low risk of liquefaction, lateral spreading, and seismic landslides.
Structural	• No deficiencies were found.
Non-Structural	• Rust and corrosion were found on the interior of the structure; they should be cleaned and repaired. Mold was also observed on the interior walls and should be cleaned. • The existing fan is not functioning. It should be repaired or replaced to provide adequate ventilation inside the structure to prevent future buildup of mold, rust and corrosion.

8.4.4.3 Strawberry Pump Station

The Strawberry Pump Station was built in 2001 and consists of a plastic cover bolted to a concrete pad on grade. The cover protects the pump and electrical panels from the weather. Table 8-9 summarizes the findings and recommendations for improvements.

Table 8-9. Strawberry Pump Station – Seismic Evaluation Summary	
Potential	Description
Seismic	• 5-10 in/sec ground shaking intensity (PGV); low risk of liquefaction, lateral spreading, and seismic landslides.
Structural	• No deficiencies were found.
Non-Structural	• None.

8.4.5 10th Avenue Reservoirs

8.4.5.1 10th Avenue Reservoir – 0.3 MG

The 10th Avenue 0.3 MG Reservoir is currently inactive due to leaks and is not providing service to the water distribution system. This reservoir is a partially buried concrete tank built in 1938 with a retrofit improvement to replace the wood framed lid with a concrete lid. Table 8-10 summarizes the findings and recommendations for improvements.

Table 8-10. 10th Avenue 0.3 MG Reservoir – Seismic Evaluation Summary	
Potential	Description
Seismic	• 5-10 in/sec ground shaking intensity (PGV); low risk of liquefaction, lateral spreading.
Structural	• Up to 4 feet earthquake-induced landslides (PGD). • Seismic landslide hazard present along the southern boundary of the City service area. A site-specific study (for slope stability) is recommended to determine the level of seismic landslide hazard. • No structural deficiencies were found.
Non-Structural	• None.

8.4.5.2 10th Avenue Reservoir – 0.7 MG

The 10th Avenue 0.7 MG Reservoir is a partially buried concrete tank built in 1951. A shotcrete cover coat was later applied on the walls. Table 8-11 summarizes the findings and recommendations for improvements.

Table 8-11. 10th Avenue 0.7 MG Reservoir – Seismic Evaluation Summary	
Potential	Description
Seismic	• 5-10 in/sec ground shaking intensity (PGV); low risk of liquefaction, lateral spreading, and seismic landslides.
Structural	• Up to 4 feet earthquake-induced landslides (PGD). • Seismic landslide hazard present along the southern boundary of the City service area. A site-specific study (for slope stability) is recommended to determine the level of seismic landslide hazard. • No structural deficiencies were found.
Non-Structural	• None.

8.4.5.3 10th Avenue Reservoir – 1.5 MG

The 10th Avenue 1.5 MG Reservoir is a partially buried concrete tank built in 1969 with a shotcrete cover coat. Table 8-12 summarizes the findings and recommendations for improvements.

Table 8-12. 10th Avenue 1.5 MG Reservoir – Seismic Evaluation Summary	
Potential	Description
Seismic	• 5-10 in/sec ground shaking intensity (PGV); low risk of liquefaction, lateral spreading.
Structural	• Up to 4 feet earthquake-induced landslides (PGD). • Seismic landslide hazard present along the southern boundary of the City service area. A site-specific study (for slope stability) is recommended to determine the level of seismic landslide hazard. • Reinforcing Steel: The amount of vertical reinforcing steel bars in the existing concrete walls is less than the recommended amount. • Wall thickness: The perimeter wall thickness exceeds the recommended limit for the unsupported height of the reservoir.
Non-Structural	• None.

8.4.6 49th Avenue Reservoir

8.4.6.1 49th Avenue Reservoir – 2.0 MG

The 49th Avenue 2.0 MG Reservoir is a prestressed reinforced concrete tank built in 1993 with a shotcrete cover coat. Table 8-13 summarizes the findings and recommendations for improvements.

Table 8-13. 10th Avenue 0.3 MG Reservoir – Seismic Evaluation Summary	
Potential	Description
Seismic	• 5-10 in/sec ground shaking intensity (PGV); low risk of liquefaction, lateral spreading.
Structural	• Up to 4 feet earthquake-induced landslides (PGD). • Seismic landslide hazard present along the southern boundary of the City service area. A site-specific study (for slope stability) is recommended to determine the level of seismic landslide hazard. • Wall thickness: The perimeter wall thickness exceeds the recommended limit for the unsupported height of the reservoir.
Non-Structural	• None.

8.4.7 General Non-Structural Considerations

It is recommended that City staff review the ASCE 41-17 Nonstructural Checklist discussed in Appendix C and consider the items at each facility for compliance with the best practices for storing items and equipment. Some conditions to consider include:

- **Fire Suppression Piping:** Make sure piping is anchored and braced in accordance with current NFPA standards. Consider anchoring and bracing all piping in all facilities.
- **Hazardous Material Storage:** Some chemicals used in the treatment process or used during regular cleaning and maintenance processes may be considered hazardous when spilled. Items storing these chemicals should be restrained to prevent displacement, tipping, or falling.
- **Hazardous Material Distribution:** Natural gas piping should be anchored or braced adequately to prevent damage that might allow the hazardous material to release.
- **Shutoff Valves:** Piping containing hazardous material, including natural gas, should have shutoff valves or other devices to prevent spills or leaks.
- **Flexible Couplings:** Hazardous material, ductwork, and piping, including natural gas piping, should have flexible couplings.
- **Light Fixtures Lens Covers:** Make sure lens covers on light fixtures are attached with safety devices and add safety devices if necessary.
- **Industrial Storage Racks:** Industrial storage racks or similar items that are more than 12 feet high should be anchored to the floor.
- **Tall Narrow Cabinets:** Cabinets, lockers, bookshelves, etc. more than 6 feet high and with height-to-depth ratios exceeding 3:1 should be anchored to the floor or wall.
- **Fall-Prone Contents:** Equipment, stored items weighing more than 20 pounds and more than 4 feet above the floor should be braced or restrained.
- **Fall-Prone Equipment:** Equipment weighing more than 20 pounds and more than 4 feet above the floor should be braced or restrained.
- **In-Line Equipment:** Equipment installed in line with a duct or piping system, with an operating weight more than 75 pounds should be laterally braced independent of the duct or piping system.
- **Tall Narrow Equipment:** Equipment, tanks, etc. more than 6 feet high and with height-to-depth ratios exceeding 3:1 should anchored to the floor or wall.
- **Suspended Equipment:** Equipment suspended without lateral bracing should be free to swing or move with the structure without damaging itself or adjoining components.
- **Heavy Equipment:** Floor-supported or platform-supported equipment weighing more than 400 pounds should be anchored to the structure.
- **Conduit Couplings:** Conduit greater than 2.5 inches should have flexible couplings.
- **Flexible Couplings:** Fluid and gas piping should have flexible couplings.
- **Fluid and Gas Piping:** Fluid and gas piping should be anchored and braced to the structure to limit spills or leaks.

Buildings may also contain some form of hazardous material. These materials will need to be dealt with on a case-by-case basis.

8.4.8 Pipeline Fragility

Most of the City backbone pipelines range from 10- to 16-inch diameter with a few 4- to 8-inch diameter pipelines. As discussed in *Chapter 2 Existing System Description*, pipes are composed of several different materials with ductile iron as the most installed (around 40% in the system), followed by PVC pipe (28%) and cast iron (20%).

Liquefaction and lateral spreading are very low in the City; consequently, the repair rate due to permanent ground deformation is considered very low and the pipes would suffer little damage.

Using the peak ground velocity (5-10 inches/second) estimated in the geotechnical evaluation, and applying it to the ALA vulnerability function, result in a very small RR value for the pipe system (less than 4), indicating the potential for little to no repair due to ground shaking.

In conclusion, ground shaking or permanent ground deformation would cause little damage to the backbone pipes. However, replacement of old pipes with new ductile iron pipe with restrained joints would further increase the seismic resilience of the water system. Restrained joints are a low cost addition to pipeline installation and should be included in the City's pipeline design and construction standards.

8.5 MITIGATION OF SEISMIC HAZARDS

As mentioned in Section 7.1, the City is following recommendations for water systems outlined in the 2013 ORP, in large part, for its Water System Resilience Plan. The 2013 ORP presents target states of recovery following a major earthquake and suggests planning for long-term goals (40- to 50-year planning horizon) for water system readiness in case of a magnitude 9.0 CSZ earthquake.

After the review of the seismic evaluation of the City water system facilities, some mitigation strategies may be considered for improving the seismic resiliency of the backbone water system:

- Pipe replacement: Replace existing CI pipes with more seismic resilient pipeline systems (lower break rates) such as welded steel pipe, DI pipe with restrained joints, Earthquake Resistant Ductile Iron Pipe (ERDIP), or HDPE pipe (AWWA-C906) or Molecularly Oriented PVC pipe (AWWA-C909).
- Site-specific slope stability analyses are recommended to be performed at the 10th Avenue and 49th Avenue Reservoir sites to determine the level of seismic landslide hazard. These site-specific evaluations are included in *Chapter 9 Capital Improvement Program*.
- Maintenance and structural upgrades should be part of the City's operating plan.
- Emergency training and exercises: Emergency training and exercises focused on earthquake scenarios can be implemented to enhance the City's emergency preparedness.

CHAPTER 9
Capital Improvement Program

This chapter presents the recommended CIP for the City's existing and future water system based on the evaluations described in *Chapter 6 Water System Analysis*, *Chapter 7 Water Treatment Plant Evaluation and Upgrades,* and *Chapter 8 Seismic Risk Assessment* of this WMP. The chapter provides a summary of the recommended capital improvement projects, along with estimates of probable construction costs. Probable construction cost estimates are developed individually for each proposed improvement project.

The recommended CIP only identifies improvements at a master planning level and does not necessarily include all required on-site infrastructure improvements. A construction contingency is included to account for the conceptual nature of improvements. Subsequent detailed design is required to determine the exact sizes and locations of the recommended improvements.

The following sections of this chapter summarize the cost estimating methodology and present the capital improvement program to address existing system deficiencies and future growth.

- Cost Estimating Assumptions
- Recommended Capital Improvement Program

9.1 COST ESTIMATING ASSUMPTIONS

Construction costs are presented in May 2023 dollars based on an ENR CCI of 13,288 (20-Cities Average). Construction costs were developed based on a combination of recent City bid results and construction costs previously estimated by West Yost for similar facilities in Oregon. An estimating contingency of 30 percent of the base construction costs is used. Markups for engineering, legal, and administrative services (ELA) during design and construction are 25 percent of the base construction costs plus the final contingency, as listed below.

- Estimating Contingency: 30 percent
- ELA Markup: 25 percent of the base construction cost plus the Estimating Contingency

The total CIP cost mark-ups are 62.5 percent of the estimated base construction costs.[1] An example of how these allowances are applied to a project with an assumed base construction cost of $1.0 million is shown in Table 9-1. As shown, the total cost of all project construction contingencies (construction, design, construction management, and administration costs) these factors result in an overall multiplier of 62.5 percent of the base construction cost.

[1] The overall mark-up is compounded: [{Base Construction Cost (1.0) + Estimating Contingency (0.3)} + ELA Markup (1.3 x 0.25 = 0.325)] = 1.625 x Base Construction Cost.

Table 9-1. Example Application of Contingency Costs and Markup		
Cost Component	Percent	Cost, dollars
Estimated Base Construction Cost before Mark-ups[a]	--	1,000,000
Estimating Contingency Costs	30	300,000
Subtotal Construction Costs		**$1,300,000**
ELA Markup	25	325,000
Estimated Total Project Cost		**$1,625,000**
(a) Assumed cost of an example project.		

For this WMP, it is assumed that recommended distribution system facilities will be developed in public rights-of-way or on public property; therefore, land acquisition costs have not been included. The estimates do not include costs for annual O&M. Suggested annual O&M budgeting line items are included separately in the CIP. A summary of the construction cost assumptions for pipeline and storage improvements are included below.

9.1.1 Pipelines

Table 9-2 presents the unit construction costs for water pipelines 6-inches through 24-inches in diameter. These unit costs are categorized by typical pipeline construction either in developed areas (e.g., in urban or suburban roads) or undeveloped areas (e.g., across open fields or in rural roads) and are representative of pipeline construction under common or normal conditions. Special or difficult conditions would increase costs significantly. The unit construction costs presented below 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.

Table 9-2. Unit Construction Costs for Pipelines[a]		
	Unit Construction Cost, dollars/linear foot[b]	
Pipeline Size	Developed Areas	Undeveloped Areas
6-inch diameter	169	115
8-inch diameter	225	154
10-inch diameter	226	192
12-inch diameter	227	174
16-inch diameter	302	231
18-inch diameter	340	260
20-inch diameter	378	289
24-inch diameter	400	314
(a) Based on May 2023 ENR CCI of 13,288 (20-Cities Average).		
(b) Estimated construction costs reflect a 10 percent reduction in bid costs to account for the current economic bidding climate.		

9.1.2 Storage Reservoirs

Table 9-3 summarizes the estimated construction costs for both above-ground concrete and steel treated water storage reservoirs between the size range of 1.0 to 3.0 MG. These costs generally include the installation of the storage reservoirs, site piping, earthwork, paving, instrumentation, and related sitework. These costs are representative of construction under normal excavation and foundation conditions and would be significantly higher for special or difficult foundation requirements.

Table 9-3. Construction Costs for Treated Water Storage Reservoirs[a]

Capacity, MG	Estimated Construction Cost, million dollars[b]	
	Above-ground Concrete	Above-ground Steel
1.0	3.0	2.4
2.0	4.0	3.3
3.0	4.9	4.0

(a) Based on May 2023 ENR CCI of 13,288 (20-Cities Average).
(b) Estimated construction costs do not reflect an adjustment to account for the current economic bidding climate.

9.1.3 Pump Stations

Pump stations will be required at ground level reservoirs to lift water to the hydraulic grade of the City's water distribution system. Estimated construction costs for reservoir pump stations, as shown in Table 9-4, are based on enclosed stations with architectural and landscaping treatment suitable for residential areas. Pump station costs can vary considerably, depending on architectural design, pumping head, and pumping capacity. Therefore, these costs presented below are representative of construction under common or normal conditions and would be significantly higher for special or difficult conditions.

Pump station cost estimates include the installation of the pumps, site piping, earthwork, paving, on site backup/standby power generator, SCADA, and related sitework.

Table 9-4. Construction Costs for Booster Pump Stations[a]

Firm Capacity, mgd[b]	Estimated Construction Cost, million dollars[c]
0.5	1.1
1	1.1
2	1.5
3	1.7

(a) Based on May 2023 ENR CCI of 13,288 (20-Cities Average).
(b) Equal to the total pumping capacity with the largest pump out of service or on standby.
(c) Estimated construction costs do not reflect an adjustment to account for the current economic bidding climate.

9.1.4 Control Valves

Two types of control valves are recommended to meet the City's operational goals and meet water system performance criteria: pressure reducing valves (PRVs) and altitude valves. PRVs are recommended for re-zoning a portion of the Main Zone to reduce system pressures. Altitude valves are recommended to regulate tank filling and prevent tank overfilling. Check valves are also recommended in some locations to provide flexibility and redundancy to move water between pressure zones during peak demands and/or emergency conditions.

The construction cost for a new control valve station (pressure reducing or altitude valve) or station upgrade is estimated to be approximately $250,000 for normal construction conditions. The construction cost for a new pressure reducing station or an existing pressure reducing station upgrade under special or difficult conditions (e.g., construction in high traffic areas) is estimated to be approximately $300,000. The construction cost for a new check valve connection is estimated to be approximately $5,000.

Construction cost estimates for a control valve station include the installation of control valve(s), a concrete utility vault, access hatches, site piping, earthwork, paving, SCADA, and related sitework.

9.2 RECOMMENDED CAPITAL IMPROVEMENT PROGRAM

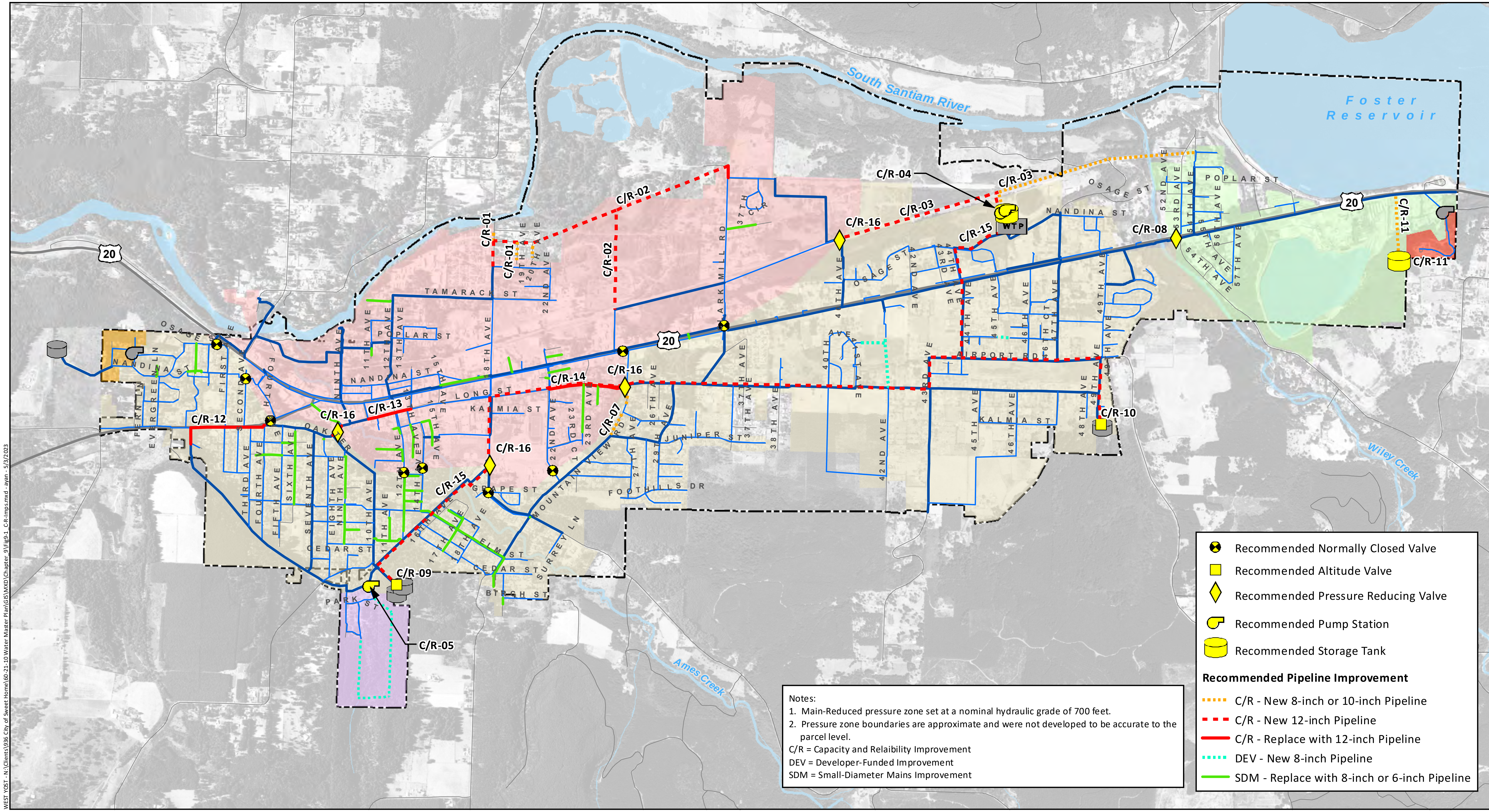
This section presents a summary of the CIP recommended to address identified deficiencies. Recommended capital improvement projects were identified as Operations and Maintenance (O&M) Improvements and Capital Improvements. Capital Improvements are subcategorized in five categories: Capacity or Reliability Improvements (C/R), Fire Flow Improvements (FFI), Small Diameter Mains Improvements (SDM), Seismic Improvements, and WTP Improvements. C/R and SDM projects are shown on Figure 9-1, and FFI projects are shown on Figure 9-2.

The locations of and justification for all proposed capacity and reliability, fire flow and small diameter main improvements are summarized in *Chapter 6 System Analysis*. WTP improvements, identified in *Chapter 7 Water Treatment Plant Evaluation and Upgrades,* and seismic improvements, identified in *Chapter 8 Seismic Risk and Mitigation Plan,* are also included in the CIP.

Some projects are deemed higher priority improvements and are identified as 5-year capital improvements. All WTP Improvements were identified as 5-year capital improvements. Capacity improvement projects identified as 5-year capital improvements are assumed to improve locations with fire flow deficiencies greater than 2,000 gpm, as shown in Figure 6-6, and locations where pressures are below 40 psi, as shown in Figure 6-3.

The 5-year CIP and 20-year CIP are presented in Table 9-5, with an estimated capital cost of $10.6M and $47.3M, respectively. The total overall CIP capital cost is approximately $57.9M as shown in Table 9-5. Pipeline replacements under the SDM Improvements should also be prioritized annually, at a projected cost of approximately $313,700 per year assuming an ongoing program over 20 years. All costs are presented in current dollars. It is recommended that the City account for future inflation by increasing the costs by 3 percent per year from 2023 dollars during preparation of the annual budget.

If funds allow, it is recommended that the City constructs CIP project C/R-15 identified in Table 9-5 as part of the 5-year CIP. Construction of C/R-15 will create dedicated fill pipelines from the proposed Main Zone PS (C/R-04) at the WTP to directly fill the 10th Avenue and 49th Avenue Reservoirs. C/R-15 will work in conjunction with the proposed altitude valve (C/R-10) (included in the 5-year CIP) at the 49th Avenue Reservoir to help simplify reservoir operations by eliminating the need to throttle flow into the 49th Avenue Reservoir to direct flow into the 10th Avenue Reservoir.



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Recommended Pressure Zones

- Strawberry
- LakePointe
- Main
- Main-Reduced (New)
- Foster (New)
- 10th Ave (New)

- Existing Water Treatment Plant
- Existing Storage Tank
- Potable Water Pump Station
- City Limit

Existing System Pipelines

- Diameter Less than 10-inches
- Diameter 10-inches and Greater

N

0 900 1,800

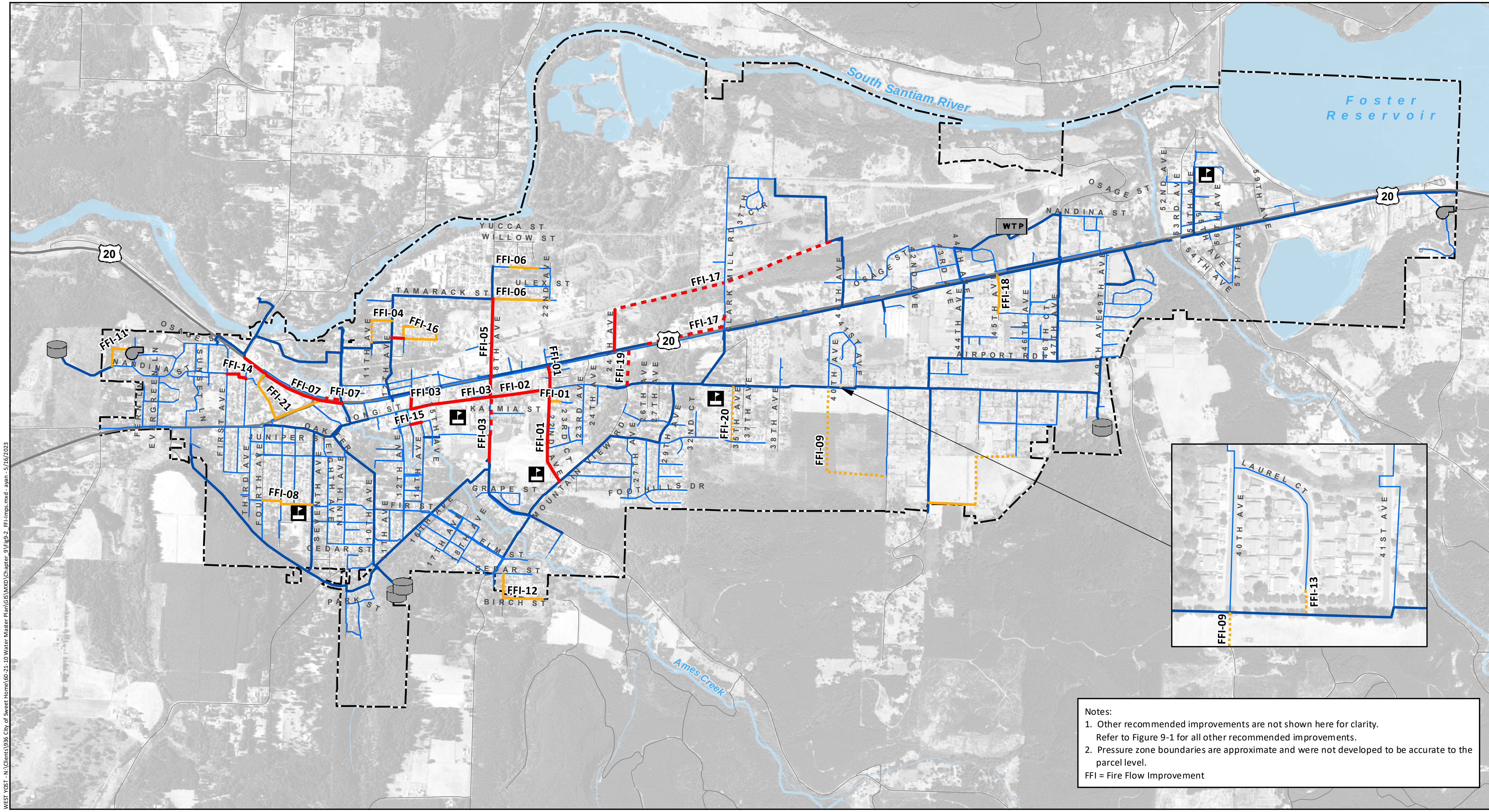
Scale in Feet

WEST YOST

DRAFT

Figure 9-1
Future System Recommended
Non-Fire Flow Improvements

City of Sweet Home
Water Master Plan



Notes:
1. Other recommended improvements are not shown here for clarity. Refer to Figure 9-1 for all other recommended improvements.
2. Pressure zone boundaries are approximate and were not developed to be accurate to the parcel level.

FFI = Fire Flow Improvement

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WTP Existing Water Treatment Plant

Existing Storage Tank

Potable Water Pump Station

Existing System Pipelines

Diameter Less than 10-inches

Diameter 10-inches and Greater

Recommended Pipeline Improvement

FFI - New 8-inch Pipeline

FFI - New 12-inch Pipeline

FFI - Replace with 8-inch or 10-inch

FFI - Replace with 12-inch

City Limit

School

N

0 900 1,800

Scale in Feet

WEST YOST

DRAFT

Figure 9-2
Future System Recommended Fire Flow Improvements

City of Sweet Home
Water Master Plan

Table 9-5. Recommended Capital Improvement Program for the City of Sweet Home[a]					
CIP ID	Improvement Type	Priority	Improvement Description	Construction Cost[b] dollars	Capital Cost[c] dollars
Operations and Maintenance					
O&M-01	WTP Operation and Maintenance	Annual	• Perform operation and maintenance projects at the WTP described in Chapter 7, Section 7.3.	-	75,000
O&M-02	Seismic Operation and Maintenance	Annual	• General Non-Structural considerations. Review and address the best-practices described in Chapter 8, Section 8.4.8. This is assumed to be an ongoing program over 20 years with an annual budget of 15,000.	-	15,000
			Annual Operations and Maintenance Total	-	**$90,000**
Capital Improvements					
Capacity or Reliability Improvements					
C/R-01	Pipeline	20-year	• Install approximately 1,250 LF of 12-inch pipeline in 18th Avenue and Willow Street. • Install approximately 850 LF of 8-inch pipeline in 18th Avenue, 19th Avenue, and 20th Avenue.	618,000	773,000
C/R-02	Pipeline	20-year	• Install approximately 6,000 LF of 12-inch pipeline to connect existing pipelines in 24th Avenue and Clark Mill Road, and future pipelines in Willow Street. • (see C/R-01). • Replace approximately 200 LF of 2-inch pipeline with 12-inch pipeline at the northern terminus of Clark Mill Road to connect to the new 12 inch pipeline.	1,402,000	1,753,000
C/R-03	Pipeline	5-year	• Install approximately 3,900 LF of 10-inch pipeline from the Foster Zone PS (see C/R-04) discharge pipelines to existing pipelines in 54th Avenue, and replace a 300 LF portion of pipeline in 4th Avenue, from Redwood Street to Quince Street.	1,048,000	1,310,000
	Pipeline	20-year	• Install approximately 3,100 LF of 12-inch pipeline from discharge pipelines at future Main Zone PS to existing pipelines in 40th Avenue). • Include a check valve connection between the two pipelines as a back-up supply to the Foster Zone from the Main Zone.	701,000	876,000
C/R-04	Storage Reservoir	20-year	• Install a new 3.0 MG at-grade reservoir and pump station at the WTP.	5,200,000	6,500,000
	Pump Station	20-year	• Approximately 0.11 mgd of firm capacity to supply the Foster Zone (to East). • Approximately 2.48 mgd of firm capacity to supply the Main Zone reservoirs (to South) via dedicated reservoir fill pipelines (see C/R-15).	2,103,000	2,629,000
C/R-05	Pump Station	20-year	• Install a new hydropneumatic pump station at the southern-most end of 10th Avenue to supply existing and future high-elevation development. Firm capacity of 1,530 gpm (Includes adequate firm capacity to provide 1,500 gpm for fire flows).	2,003,000	2,504,000
C/R-06	Control Valve	20-year	• Install four (4) PRVs and close nine (9) valves to create the new Reduced Pressure Zone, set to HGL 700 ft to decrease existing high pressures (> 80 psi). a) PRV along 10-inch pipeline in Terrace Lane, between Long Street and Oak Ter. This PRV is closed under normal conditions. b) PRV along 10-inch pipeline near 873 18th Avenue. This PRV is open under normal conditions. c) PRV along future 12-inch pipeline (see C/R-14), near 2851 Long Street. This PRV is closed under normal conditions. d) PRV along 10-inch pipeline along the railroad and immediately west of 40th Avenue. This PRV is open under normal conditions.	1,300,000	1,625,000
C/R-07	Pipeline	20-year	• Install approximately 900 LF of 8-inch pipeline in Mountain View Road to connect existing pipelines in Juniper Street, Kalamia Street, and Long Street.	263,000	329,000
C/R-08	Control Valve	5-year	• Install a new PRV along the existing 16-inch in the Santiam Hwy, east of the Wiley Creek crossing, to provide a redundant/emergency connection to the proposed Foster Zone.	325,000	406,000
C/R-09	Control Valve	20-year	• Install a new altitude valve at the 10th Avenue Reservoirs to regulate inflows. This should be paired with a check valve on the outflow pipeline for unrestricted flow into the distribution system. Construct valving so that future reservoir fill pipeline (see C/R-15) can be connected and abandon the existing 10-inch cast iron pipeline.	325,000	406,000
C/R-10	Control Valve	5-year	• Install a new altitude valve at the 49th Avenue Reservoir to regulate inflows. This should be paired with a check valve on the outflow pipeline for unrestricted flow into the distribution system. Construct valving so that future reservoir fill pipeline (see C/R-15) can be connected.	325,000	406,000
C/R-11	Storage Reservoir	5-year	• Install a new 800 kgal storage reservoir to serve the proposed Foster Zone (HGL 775 ft).	2,886,000	3,608,000
	Pipeline	5-year	• Install approximately 1,300 LF of 10-inch pipeline to connect the reservoir to existing pipelines in the Santiam Hwy.	382,000	478,000
C/R-12	Pipeline	20-year	• Replace approximately 1,750 LF of 8-inch and 6-inch pipeline with 12-inch in Vista Lane and Halsey-Sweet Home Hwy. This helps build out the transmission network by connecting existing and/or future transmission pipelines.	516,000	645,000
C/R-13	Pipeline	20-year	• Replace approximately 850 LF of 6-inch pipeline with 12-inch in Long Street, from 10th Avenue to 13th Avenue. This helps build out the transmission network by connecting existing and/or future transmission pipelines.	251,000	314,000

Table 9-5. Recommended Capital Improvement Program for the City of Sweet Home[a]

CIP ID	Improvement Type	Priority	Improvement Description	Construction Cost[b] dollars	Capital Cost[c] dollars
C/R-14	Pipeline	20-year	• Replace approximately 1,500 LF of 4-inch and 6-inch pipeline with 12-inch in Long Street, from 22nd Avenue to Mountain View Road. This helps build out the transmission network by connecting existing and/or future transmission pipelines.	443,000	554,000
C/R-15	Pipeline	20-year	• Install approximately 22,000 LF of 16-inch pipeline to create dedicated fill pipelines from the proposed Main Zone PS at the WTP (see C/R-04) to the Main Zone Reservoirs.	8,637,000	10,796,000
			Capacity Improvements Subtotal	**$28,728,000**	**$35,912,000**
Fire Flow Improvements					
FFI-01	Pipeline	5-year	• Replace approximately 2,300 LF of 6-inch pipelines in 22nd Avenue with 12-inch, from Santiam Hwy to Mountain View Road to improve fire flow to the Junior High School (5,500 gpm required). • Replace 200 LF of existing 6-inch pipeline in Kalmia Street with 8-inch, up to the existing hydrant (2,000 gpm required).	737,000	921,000
FFI-02	Pipeline	20-year	• Replace approximately 1,200 LF of 4-inch pipeline in Long Street with 12-inch, from 18th Avenue to 22nd Avenue to improve fire flow to the nearby Junior High and High Schools. This improvement also builds out the transmission network.	354,000	443,000
FFI-03	Pipeline	5-year	• Replace approximately 3,500 LF of 4-inch, 6-inch, and 8-inch pipelines with 12-inch in 13th Avenue from Santiam Hwy to Long Street, Long Street from 13th Avenue to 18th Avenue, and 18th Avenue from Santiam Hwy to 873 18th Avenue, to improve fire flow to the nearby Junior High and High Schools. This improvement also builds out the transmission network.	1,033,000	1,291,000
FFI-04	Pipeline	20-year	• Install approximately 450 LF of 8-inch pipeline in 11th Avenue from Poplar Street to Redwood Street. • Replace approximately 400 LF of 4-inch pipeline in Redwood Street with 8-inch pipeline.	249,000	311,000
FFI-05	Pipeline	20-year	• Replace approximately 1,500 LF of existing 6-inch pipeline with 12-inch in 18th Avenue from Tamarack Street to Santiam Hwy to improve light industrial and commercial fire flows (3,000 gpm required).	443,000	554,000
FFI-06	Pipeline	20-year	• Replace approximately 500 LF of 4-inch pipeline with 8-inch in Vine Street east of 18th Avenue. • Replace approximately 1,100 LF of 6-inch pipeline with 8-inch in Tamarack Street east of 18th Avenue.	468,000	585,000
FFI-07	Pipeline	20-year	• Replace approximately 2,100 LF of 6-inch pipeline in Santiam Hwy with 12-inch between Pleasant Valley Road and 9th Avenue. • Install approximately 400 LF of 12-inch pipeline in Santiam Hwy to loop pipelines on both sides of Santiam Hwy. • These improvements increase fire flow in the commercial highway area (3,000 gpm required) and build out the transmission network.	738,000	923,000
FFI-08	Pipeline	5-year	• Replace approximately 350 LF of 4-inch and 6-inch pipeline with 10-inch in Elm Street from 6th Avenue to 7th Avenue. • Replace approximately 700 LF of 4-inch pipeline with 8-inch in Elm Street from 4th Avenue to 6th Avenue. • These improvements increase fire flow to Oak Heights Elementary (4,000 gpm required).	308,000	385,000
FFI-09	Pipeline	20-year	• Install approximately 2,800 LF of 8-inch pipeline to loop a long dead end pipeline in 42nd Avenue with 12-inch pipelines in Long Street.	561,000	701,000
FFI-10	Pipeline	20-year	• Replace approximately 900 LF of 6-inch pipeline with 8-inch in Coulter Lane. • Install approximately 1,700 LF of 8-inch pipeline to loop dead ends in Coulter Lane and 46th Avenue. • These improvements increase fire flows locally where pressures are low (high elevations) under normal conditions.	521,000	651,000
FFI-11	Pipeline	20-year	• Replace approximately 800 LF of 6-inch pipeline with 8-inch in Strawberry Ridge and Strawberry Loop to improve fire flow in the Strawberry Zone (1,500 gpm required).	234,000	293,000
FFI-12	Pipeline	20-year	• Replace approximately 1,200 LF of 6-inch pipeline with 8-inch in 23rd Avenue and Birch Street.	351,000	439,000
FFI-13	Pipeline	20-year	• Install approximately 80 LF of 8-inch pipeline to connect the dead-end in Laurel Ct to existing pipelines in Long Street.	23,000	29,000
FFI-14	Pipeline	20-year	• Replace approximately 450 LF of 6-inch pipeline with 12-inch between 1st Avenue and 2nd Avenue and east of Nandina Street (pipeline crosses through private properties) to improve fire flows in 2nd Avenue (3,000 gpm required).	133,000	166,000
FFI-15	Pipeline	20-year	• Replace approximately 250 LF of 6-inch and 8-inch pipeline with 12-inch in Kalmia Street to improve fire flows locally (3,000 gpm required).	74,000	93,000
FFI-16	Pipeline	20-year	• Replace approximately 250 LF of 6-inch pipeline with 12-inch in Poplar Street from 12th Avenue to 13th Avenue. • Replace approximately 1,700 LF of 4-inch and 6-inch pipeline with 8-inch in 1th Avenue, Poplar Street, and Quince Street loop. • These improvements increase fire flows to the loop (2,000 gpm required).	571,000	714,000

Table 9-5. Recommended Capital Improvement Program for the City of Sweet Home[a]

CIP ID	Improvement Type	Priority	Improvement Description	Construction Cost[b] dollars	Capital Cost[c] dollars
FFI-17	Pipeline	20-year	• Install approximately 4,500 LF of 12-inch pipeline parallel to the railroad to connect loop pipelines in 24th Avenue and Clark Mill Road, and north of 40th Avenue. • Install approximately 1,700 LF of 12-inch pipeline in Santiam Hwy to loop pipelines in 24th Avenue and Clark Mill Road. This pipeline is required to provide looping once the Reduced zone is created, which will isolate previously looped pipelines. • Replace approximately 800 LF of 6-inch pipeline with 12-inch in 24th Avenue, north of Santiam Hwy, to connect transmission pipelines. • These improvements also build out the transmission network.	2,066,000	2,583,000
FFI-18	Pipeline	20-year	• Replace approximately 750 LF of 6-inch pipeline with 8-inch in 45th Avenue from Santiam Hwy to Airport Lane to improve fire flows locally (3,000 gpm required).	219,000	274,000
FFI-19	Pipeline	20-year	• Install approximately 700 LF of 12-inch pipeline between Santiam Hwy and Long St to loop the two pipelines which will become isolated dead-ends when the area is re-zoned.	207,000	259,000
FFI-20	Pipeline	20-year	• Install approximately 1,100 LF of 8-inch pipeline in 35th Avenue, between Long Street and Juniper Street.	322,000	403,000
FFI-21	Pipeline	20-year	• Replace approximately 2,000 LF of 4-inch pipeline in 4th Avenue and Halsey-Sweet Home Hwy, and loop this new pipeline at both ends with existing pipelines in the Santiam Hwy.	585,000	731,000
FFI-22	Pump Station	20-year	• Install an additional 660 gpm of additional firm capacity to the Lake Pointe pump station.	650,000	813,000
			Fire Flow Improvements Subtotal	**$10,847,000**	**$13,562,000**
Small Diameter Mains Improvements					
SDM-01	Pipeline	20-year	• Replace all small-diameter mains (defined as 3-inch or smaller in diameter) with 6-inch for dead-ends. Approximately 8,600 LF of dead-end small-diameter mains in the City.	1,889,000	2,361,000
SDM-02	Pipeline	20-year	• Replace all small-diameter mains (defined as 3-inch or smaller in diameter) with 8-inch for looped pipelines. Approximately 10,700 LF of looped small-diameter mains in the City.	3,130,000	3,913,000
			Small Diameter Mains Improvements Subtotal	**$5,019,000**	**$6,274,000**
Seismic Improvements					
SEI-01	Seismic Structural Improvements	20-year	• Address the seismic structural deficiencies at the WTP building.	-	250,000
SEI-02	Stope Stability Analysis	20-year	• Perform site-specific slope stability analyses at the 10th Avenue and 49th Avenue reservoir sites to determine the level of seismic landslide hazards. Refer to Chapter 8, Section 8.5.	-	60,000
			Seismic Improvements Subtotal	**-**	**$310,000**
Water Treatment Plant Improvements					
WTP-01	WTP Improvements	5-year	• Filter feed piping manifold system	-	77,000
WTP-02	WTP Improvements	5-year	• New WTP standby generator and automatic transfer switch	-	984,000
WTP-03	WTP Improvements	5-year	• Filter sludge removal system replacement	-	750,000
WTP-04	WTP Improvements	5-year	• New sludge drying bed	-	33,000
			Water Treatment Plant Improvements Subtotal	**-**	**$1,844,000**
			5-year Capital Improvement Program Total	7,044,000	10,649,000
			20-year Capital Improvement Program Total	37,550,000	47,253,000
			Capital Improvement Program Total	**$44,594,000**	**$57,902,000**

(a) Costs are rounded to the nearest thousand dollars. Improvements in this table are considered "backbone" improvements. Smaller, in-tract, improvements are not included and are assumed to be constructed by future development proponents.
(b) Construction cost is equal to the base construction cost with a 30 percent estimating contingency.
(c) Capital cost is equal to the construction cost with a 25 percent markup for engineering, legal, and administrative services.

Appendix A

Hydrant Testing Plan Memorandum

5 Centerpointe Drive
Suite 130
Lake Oswego OR 97035

503.451.4500 phone
530.756.5991 fax
westyost.com

MEMORANDUM

DATE: December 9, 2021

Project No.: 936-60-21-10
SENT VIA: EMAIL

TO: Greg Springman
Trish Rice
Steven Haney
Dominic Valloni

FROM: Kambria Tiano, PE (CA) RCE #84129
Nick Szigeti, EIT (OR) #96476EI

REVIEWED BY: Sandrine Ganry, PE (OR) #80326PE

SUBJECT: Hydrant Testing Plan – City of Sweet Home Water Master Plan

This memorandum summarizes the proposed hydrant testing and pressure data collection required to calibrate and validate the City of Sweet Home's (City) hydraulic model of the existing water system. West Yost's recommended program for hydrant flow testing is summarized below and provided for your review and comment. Details related to the hydrant testing program are discussed in this memorandum and organized as follows:

- Hydrant Testing Program Overview
- Personnel and Water System Data Requirements
- Testing Requirements and Procedure
- Summary of Hydrant Testing

Supplemental information pertinent to data collection in the field are provided in the following attachments:

- Attachment A: Hydrant Test Location Maps
- Attachment B: Hydrant Test Data Tables

Hydrant Testing Program Overview

Hydrant fire flow tests will be used to "spot-check" system pressures and verify that the City's hydraulic model accurately predicts fire flow conditions in the existing water system. These tests will help confirm that the hydraulic model can simulate observed fire flows and pressures with no valves closed within the water system.

The hydrant tests will also validate the pipeline roughness factors (C-factors) that have been assigned to pipelines in the City's hydraulic model. Though the hydrant testing program identified in this memorandum will not isolate and test specific pipelines of known diameter and material types, calibration of the hydraulic model against the observed fire flows will provide a confirmation that assigned pipeline C-factors are adequate under high flow conditions. Approximate pipeline C-factors were updated according to pipeline diameter and material type, as provided in the City's GIS pipeline shapefile or based on correspondence with City staff, during development of the City's Small Diameter Main Replacement Program. Pipeline roughness factors were assigned based on calibrated C-factors sourced from West Yost's C-factor database[1].

Each hydrant test requires that City staff record static pressures at the test and observation hydrants, fully open the test hydrant, record flow and residual pressure at the test hydrant, record residual pressures at nearby observation hydrants, and close the test hydrant. Flow testing procedure is discussed in further detail in *Testing Requirements and Procedure*, below.

Personnel and Water System Data Requirements

West Yost would like to request the following City personnel and system data to perform the recommended hydrant testing program:

- Four (4) City staff members to perform the following:
  - Setting up and flowing the test hydrant (1 City staff)
  - Reading and recording hydrant pressure and flow data (3 City staff)
  - Dechlorination at the flowing test hydrant
  - Directing and controlling traffic as necessary to accommodate the quantities of hydrant flow that will be discharged into the street and storm drainage system during each test
- Water system Supervisory Control and Data Acquisition (SCADA) data during the period that hydrant flow testing is performed that includes the following:
  - Tank levels (water surface elevations)
  - Booster pump station (including treatment plant) flows and pressures
  - Pressure regulating valve (PRV) flows and pressures
  - Data should be provided in one-minute intervals during hydrant testing days, if possible
- Water system facility operation settings, if not indicated in the SCADA data, including:
  - Pressure setpoints for PRV or VFD-equipped pumps

Testing Requirements and Procedure

West Yost would like the City to conduct 18 hydrant tests within the City's existing water service area. Table 1 lists the locations of the proposed tests, and each test location is illustrated on Figure 1. The selected tests are distributed throughout the existing water service area, and hydrant tests were selected based on proximity to pressure zone boundaries and water system facilities, surrounding pipeline characteristics (i.e., diameter, material, age), and regions with high elevations or remote (hydraulically distant from supply) locations. Detailed location maps of each hydrant test are provided in Attachment A.

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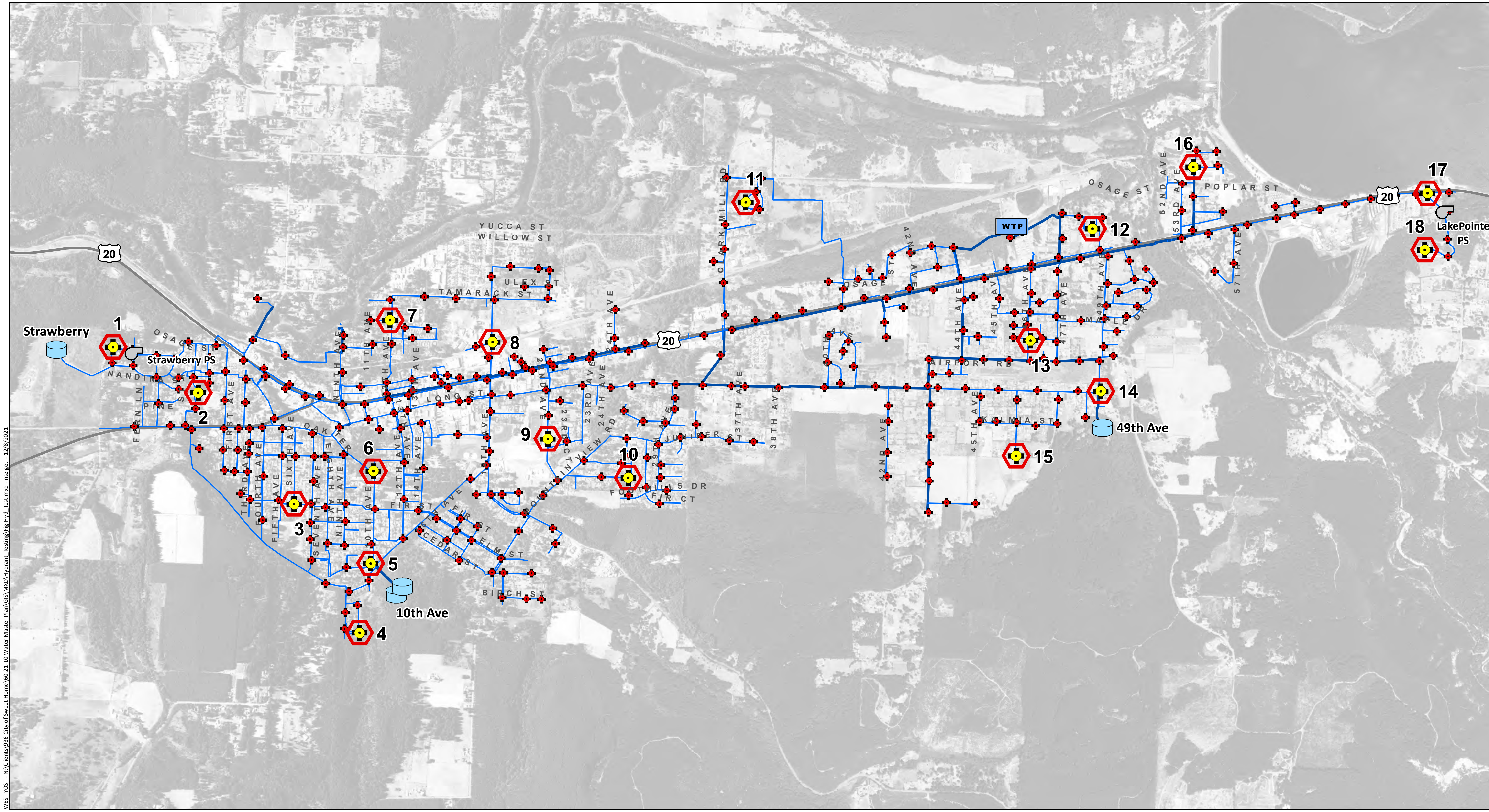
[1] West Yost's C-factor database summarizes results from over 330 uni-directional style hydrant tests. The database provides calibrated pipeline roughness factors for a variety of pipeline diameters and material types, including cast iron (over 50 hydrant tests), ductile iron (over 40 tests), and PVC (over 40 tests).

Table 1. Hydrant Test Locations		
Hydrant Test No.	Approximate Location	Comments
1	1459 Strawberry Ridge	Strawberry Pressure Zone
2	1321 Sunset Lane	High elevation
3	610 Elm Street (across from Oak Heights Elementary)	High elevation **Be mindful of school drop off/ pick-up times for this location**
4	Corner at Taylor Creek Drive and Timber Street	High elevation; dead end
5	960 Alder Street (intersection of 10th Avenue and Alder Street)	Downstream of 10th Avenue tanks
6	745 10th Avenue	1950's 10-inch cast iron
7	1806 12th Avenue	Isolated area
8	1621 18th Avenue (near railroad tracks)	1940's 6-inch cast iron
9	951 22nd Avenue	1960's-1970's 6-inch ductile iron
10	778 27th Avenue	1970's-2000's 6 to 8-inch ductile iron
11	1941 37th Circle	1980's-2000's 8-inch ductile iron
12	4879 48th Loop	Near water treatment plant
13	1219 46th Avenue	8-inch PVC
14	1199 49th Avenue	Downstream of 49th Avenue tank
15	1083 46th Avenue (at bend in 46th Avenue)	1980's 6 to 8-inch ductile iron
16	1702 54th Avenue	Isolated area
17	Intersection of Highway 20 and Riggs Hill Road	At end of long dead-end main
18	6309 LakePointe Way (in cul-de-sac)	LakePointe Pressure Zone



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WTP Water Treatment Plant

Storage Tank

Pump Station

Test Hydrant

Hydrants

Potable Water Pipelines

Less than 12-inch

12-inch and Greater

N

0 900 1,800

Scale in Feet

WEST YOST

Figure 1

Hydrant Test Location Overview

City of Sweet Home

Water Master Plan

Testing Procedure

Each test will involve maintaining flow from a single hydrant, while monitoring the residual pressure at two to three observation hydrants located near the flowing hydrant. The field-observed static and residual pressure readings will then be used to verify or calibrate the hydraulic model to observed conditions.

At least one (1) City staff member will be required at the flowing test hydrant and up to three (3) additional City personnel will be required in the field to measure static and residual pressures at the nearby observation hydrants (refer to Attachment A). Data will be recorded in the data log tables provided as Attachment B.

The general testing procedure at each of the test locations is outlined below and illustrated on Figure 2:

Step 1. Before the test, slowly open the test (flowing) hydrant and each observation hydrant to flush out possible accumulated sediments, and then close the hydrant valve before attaching the pressure gage. This allows sediments, which might damage the gage or cause faulty readings, to be flushed out from the hydrant.

Step 2. Attach the pressure gage to the hydrant with the gage's test cock valve **<u>open</u>**. Slowly open the hydrant and bleed off the gage with the gage's test cock until the hydrant is fully pressurized.

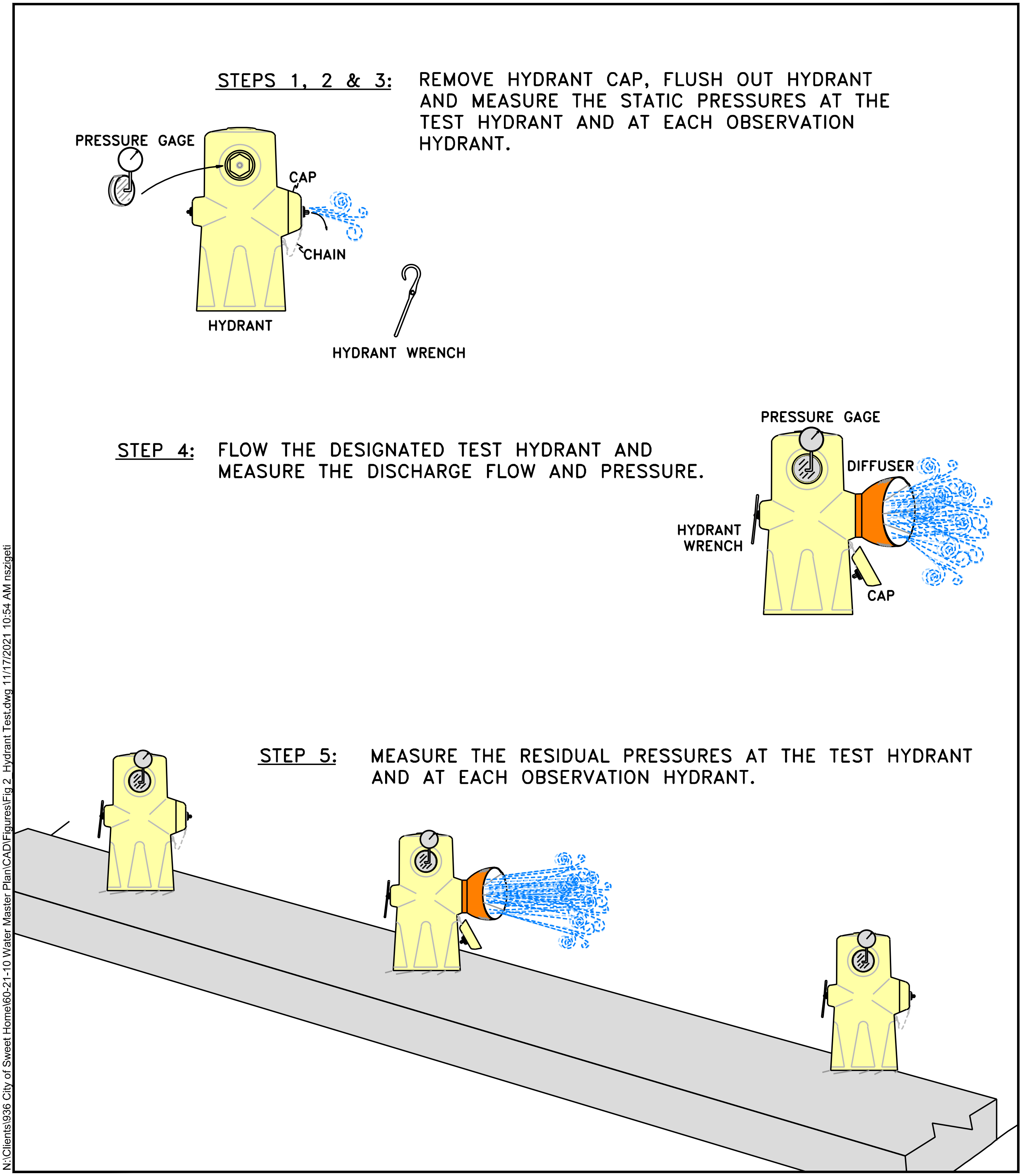
Step 3. Close the gage test cock valve, and then measure the static pressures at the designated test hydrant and each observation hydrant.

Step 4. Flow the designated test hydrant and measure the discharge flow and pressure. If system pressure at any hydrant approaches 20 pounds per square inch (psi), reduce flow from the test hydrant to maintain approximately 20 psi and note in the data log.

Step 5. Once the test hydrant flow and residual pressure have reached approximate equilibrium, measure the residual pressures at the designated test hydrant and at each observation hydrant while the test hydrant is flowing (directions should be provided via handheld radio from the City staff monitoring the test hydrant of when to record static and residual hydrant pressures).

Step 6. Continue monitoring pressure until flow and pressure has been recorded at all hydrants in the test. Record the static pressure and then detach the pressure gage. ***<u>IMPORTANT</u>****<u>: Before closing the hydrant, be sure the gage's test cock valve is open and bleeding while the hydrant is being closed.</u>*

It is anticipated that each test should take no more than thirty (30) minutes and that each hydrant will be flowing for no more than ten (10) minutes during a test.



N:\Clients\936 City of Sweet Home\60-21-10 Water Master Plan\CAD\Figures\Fig 2 Hydrant Test.dwg 11/17/2021 10:54 AM nszigeti

LEGEND

NOT TO SCALE

Figure 2

Hydrant Test Procedure

City of Sweet Home
Water Master Plan

Testing Equipment and Responsibilities

The City will be responsible for providing the necessary equipment required to perform the hydrant testing procedure described in this memorandum. Required testing equipment includes:

- Hydrant wrenches (4 minimum)
- Hydrant pressure gages (4 minimum; 5-6 preferred in case of equipment failure)
- Hydrant diffuser with pitot assembly for measuring and directing hydrant flow (preferred) or hand-held flow meter
- Two-way portable communication for each of the testing personnel
- Dechlorination tablets for hydrant runoff

The City is also responsible for notifying other City staff and residents about the scheduled hydrant testing; obtaining any approvals that may be required, providing proper drainage of the hydrant flow, and providing equipment (e.g., dechlorination) and personnel for traffic control, if required.

West Yost requests that City operations staff review and inspect each of the proposed test locations before the testing date to identify any potential problems or hazards with the selected locations. Of particular concern is the potential for flooding landscaping, building basements, or creating hazardous traffic conditions. West Yost recommends that all drainage inlets/manholes be inspected near the testing sites to confirm proper drainage.

Summary of Hydrant Testing

Hydrant testing will be performed as described above and should be completed during typical weekday demand conditions (i.e., Tuesday through Thursday). The City is responsible for conducting the hydrant testing, recording pressure and flow results, and notifying other City staff and local residents/businesses about the hydrant testing, as needed.

Hydrant testing should be completed and results recorded (see Attachment B) and provided to West Yost by **Friday, January 14, 2022**. Completion of hydrant testing by this date will ensure the Water Master Plan project remains on-schedule.

West Yost is available for a conference call with City staff prior to the scheduled testing day, if desired, to review and finalize preparations for the hydrant testing. If any questions arise regarding the procedure or required equipment, please feel free to contact Kami Tiano at (925) 425-5625 or ktiano@westyost.com.

Attachment A

Hydrant Test Locations

WEST YOST - N:\Clients\936 City of Sweet Home\60-21-10 Water Master Plan\GIS\MXD\Hydrant_Testing\FigBook-Hyd_Test.mxd - nszigeti - 12/8/2021

Flowing Hydrant
Observation Hydrant
Hydrant
Pump Station
Storage Tank
Pipeline

N

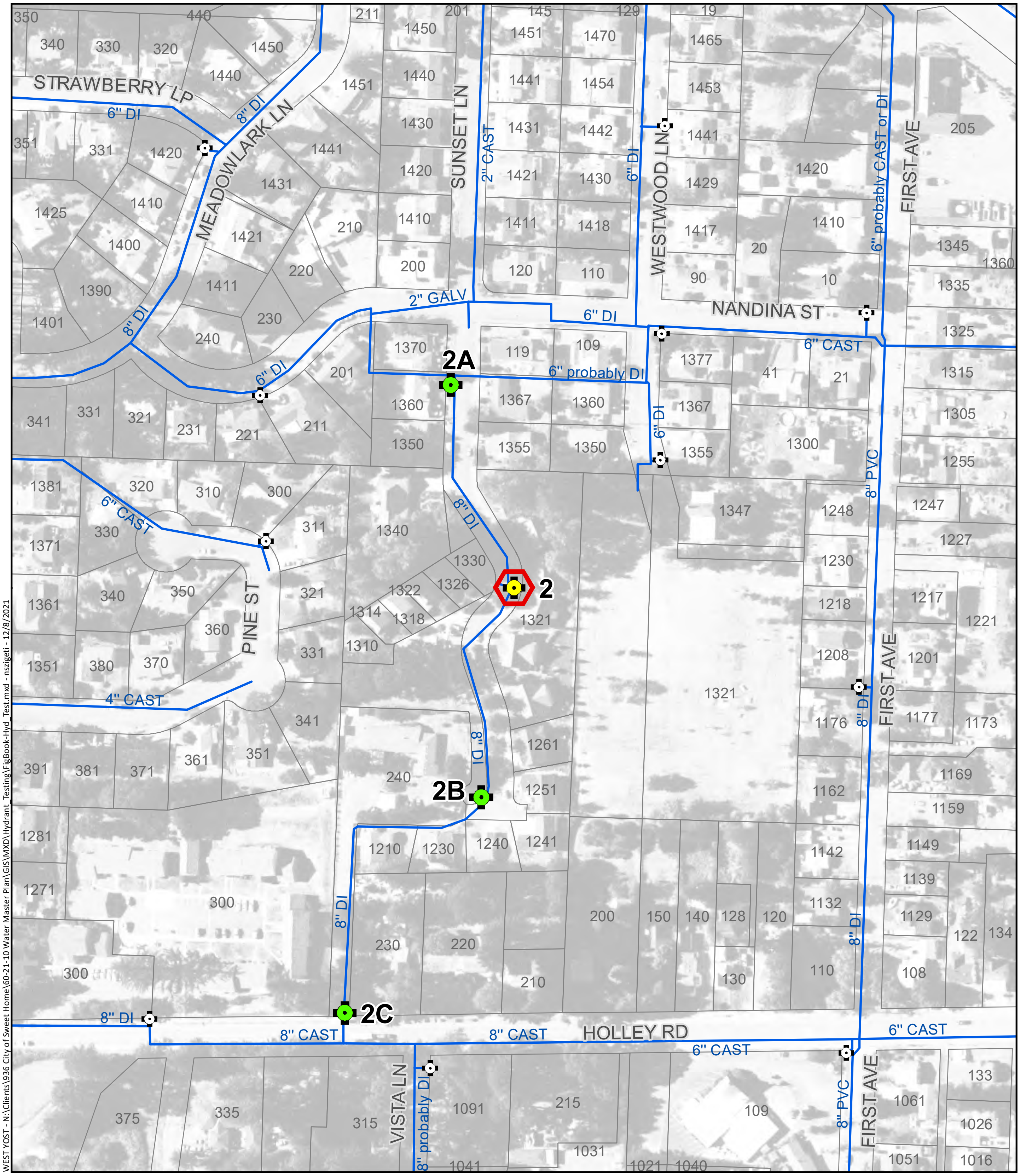
0 100 200
Scale in Feet

WEST YOST

Figure A-1

Test 1

City of Sweet Home
Water Master Plan



Flowing Hydrant

Observation Hydrant

Hydrant

Pump Station

Storage Tank

Pipeline

N

0 100 200

Scale in Feet

WEST YOST

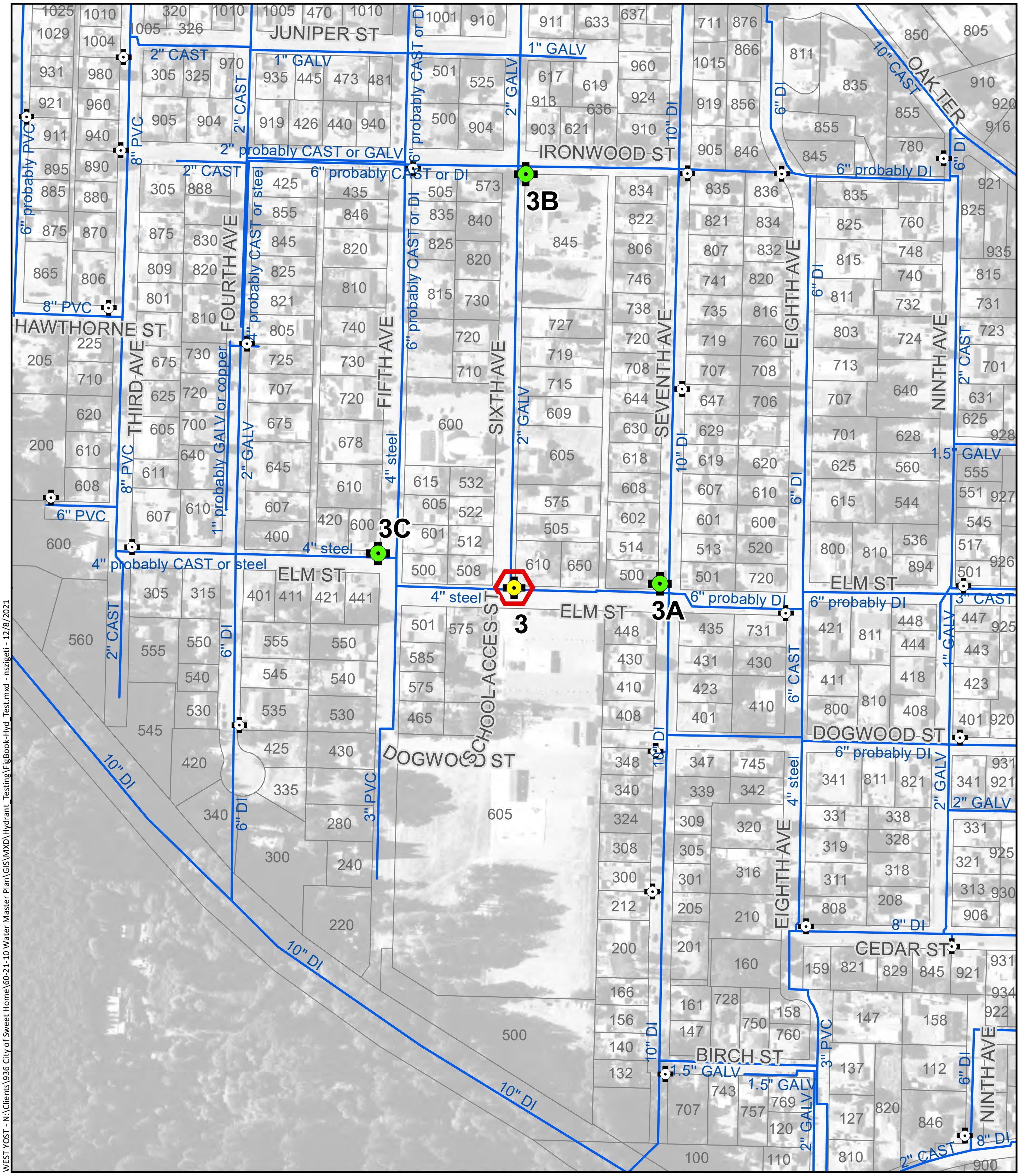
Figure A-2

Test 2

City of Sweet Home

Water Master Plan

WEST YOST - N:\Clients\936 City of Sweet Home\60-21-10 Water Master Plan\GIS\MXD\Hydrant_Testing\FigBook-Hyd_Test.mxd - nszigeti - 12/8/2021



Flowing Hydrant

Observation Hydrant

Hydrant

Pump Station

Storage Tank

Pipeline

N

0 150 300

Scale in Feet

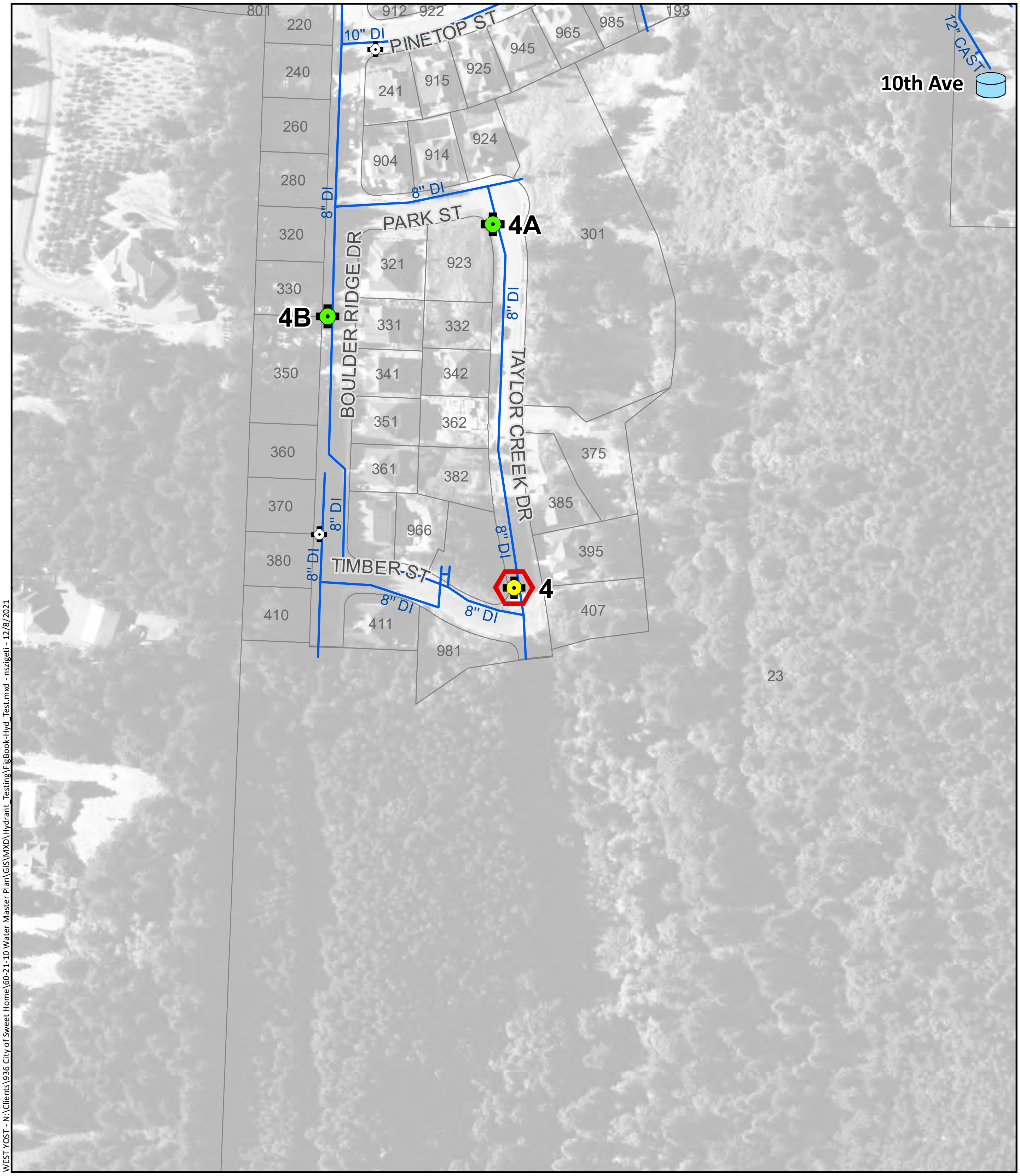
WEST YOST - N:\Clients\936 City of Sweet Home\60-21-10 Water Master Plan\GIS\MXD\Hydrant Testing\FigBook-Hyd_Test.mxd - nszigeti - 12/8/2021

Figure A-3

Test 3

City of Sweet Home

Water Master Plan



Flowing Hydrant

Observation Hydrant

Hydrant

Pump Station

Storage Tank

Pipeline

0 100 200

Scale in Feet

N

WEST YOST - N:\Clients\936 City of Sweet Home\60-21-10 Water Master Plan\GIS\MXD\Hydrant_Testing\FigBook-Hyd_Test.mxd - nszigeti - 12/8/2021

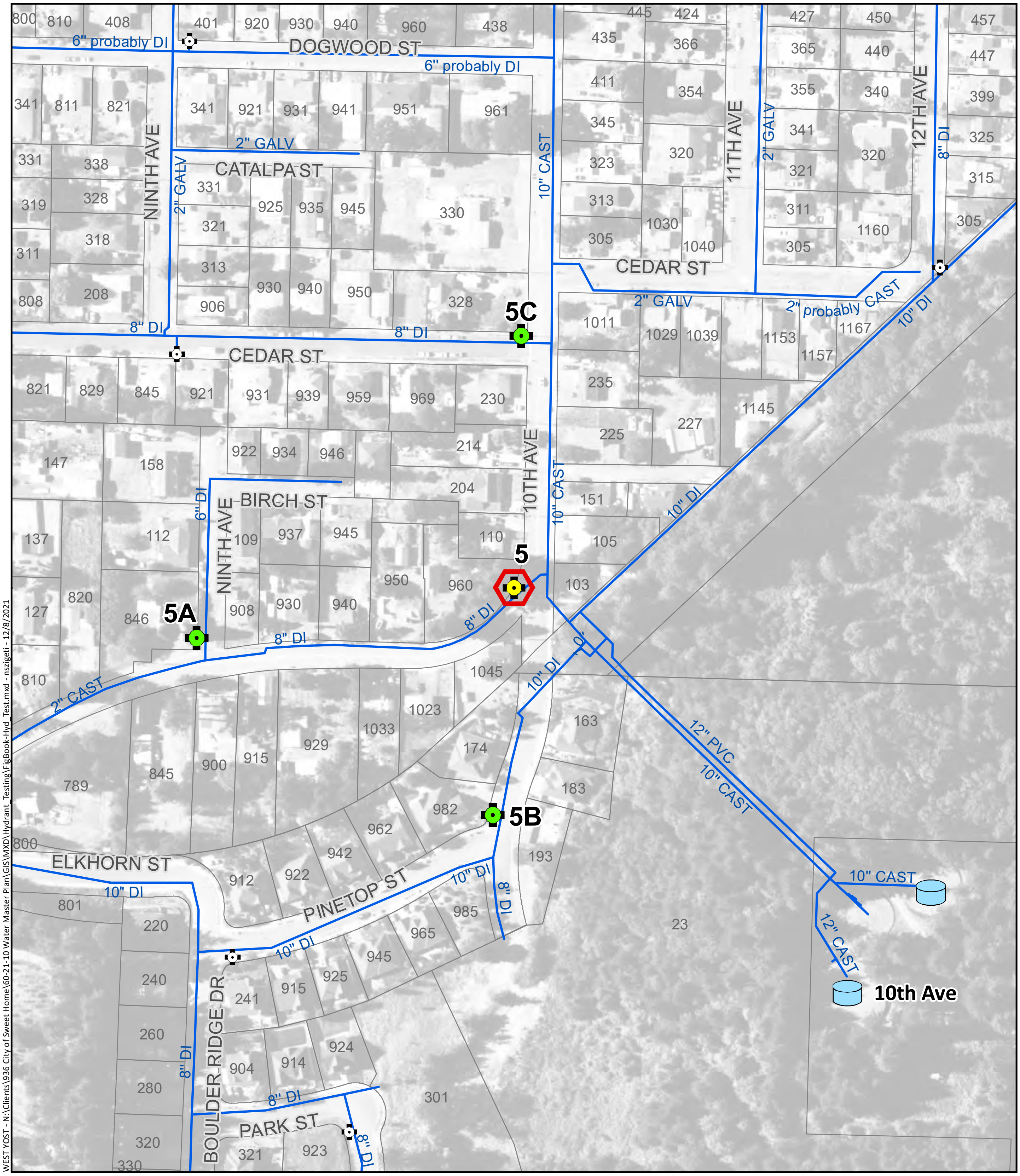
WEST YOST

Figure A-4

Test 4

City of Sweet Home

Water Master Plan



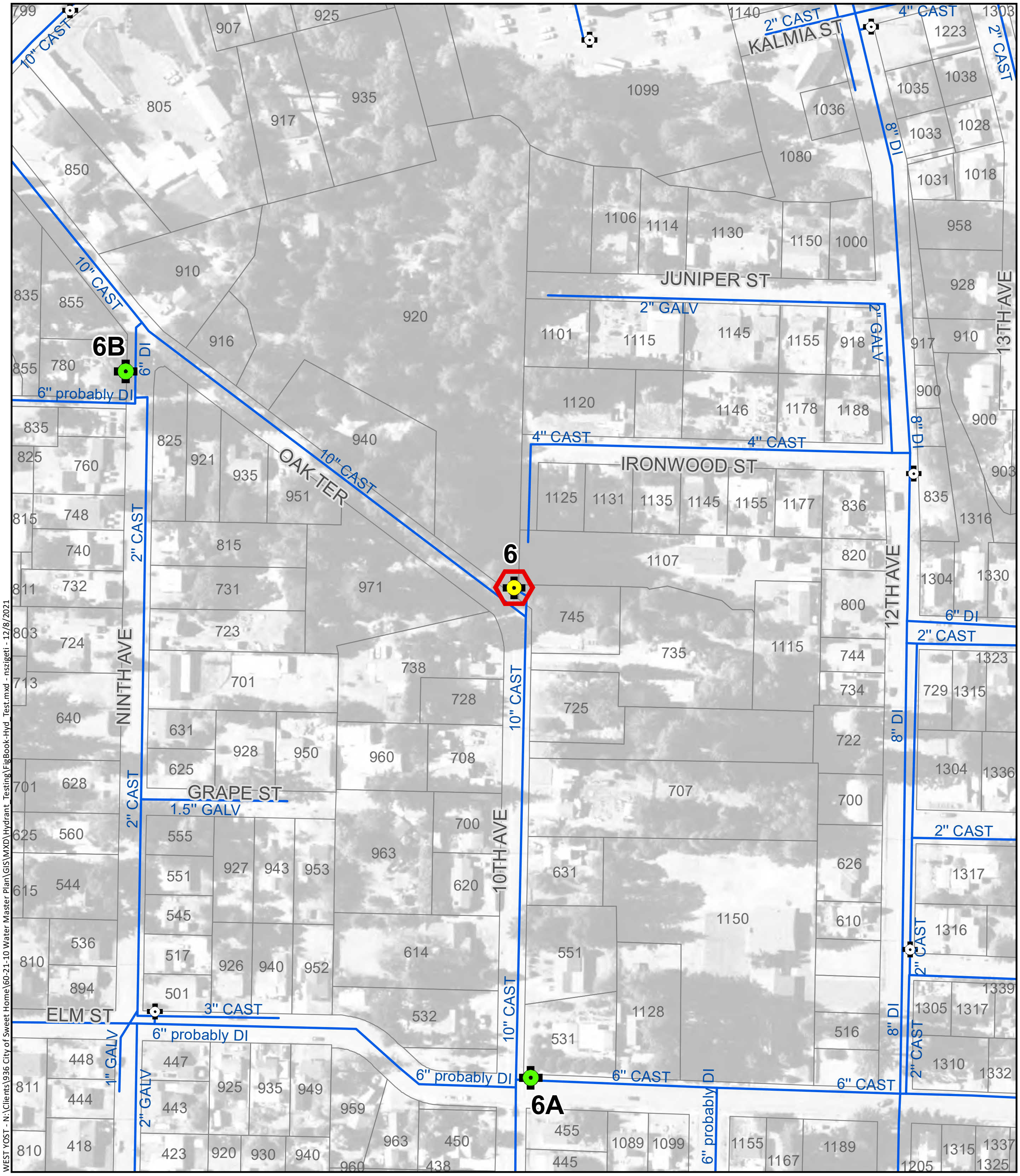
Flowing Hydrant
Observation Hydrant
Hydrant
Pump Station
Storage Tank
Pipeline

0 100 200
Scale in Feet

Figure A-5

Test 5

City of Sweet Home
Water Master Plan



Flowing Hydrant

Observation Hydrant

Hydrant

Pump Station

Storage Tank

Pipeline

N

0 100 200

Scale in Feet

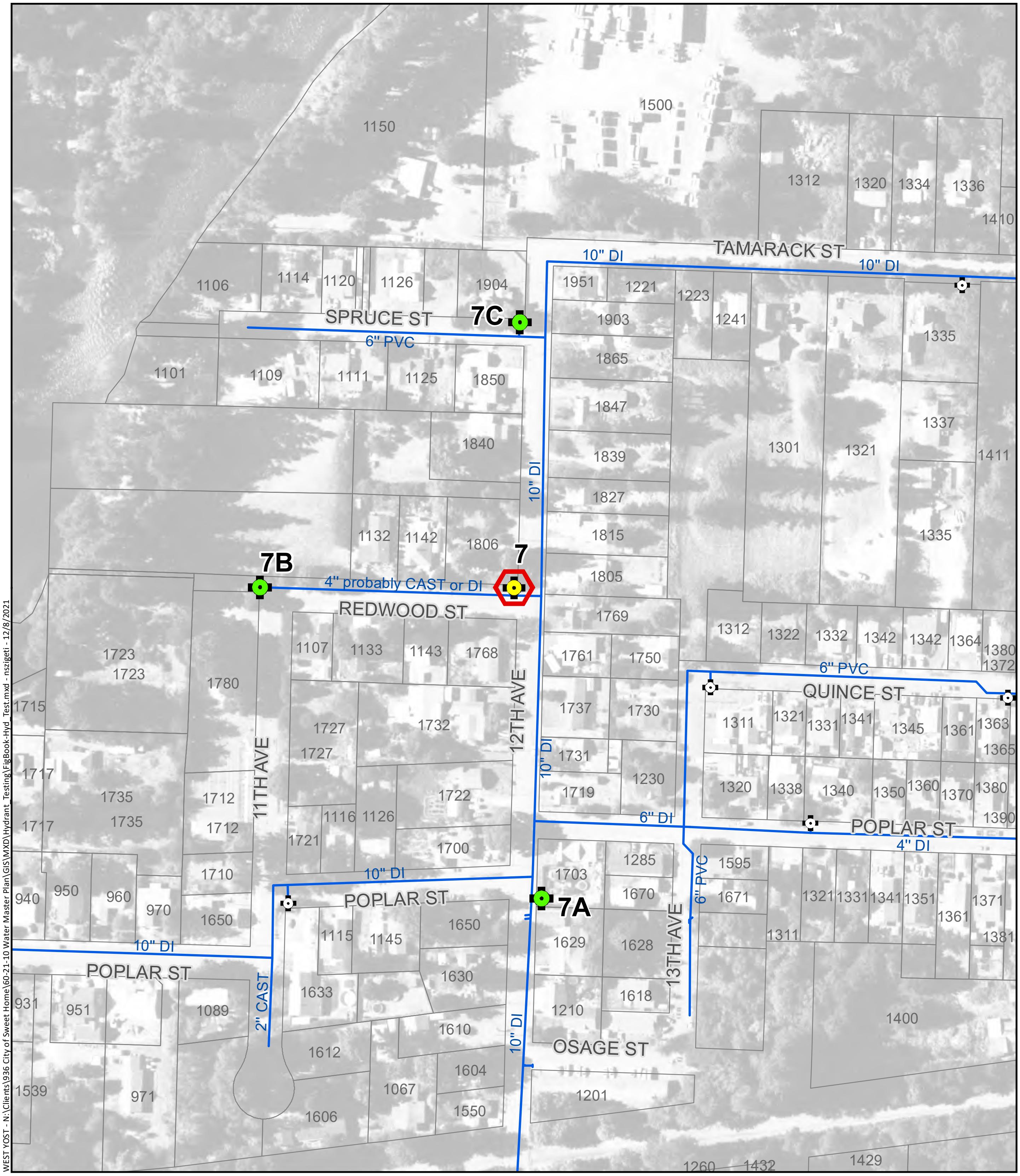
WEST YOST

Figure A-6

Test 6

City of Sweet Home

Water Master Plan



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Flowing Hydrant

Observation Hydrant

Hydrant

Pump Station

Storage Tank

Pipeline

N

0 100 200

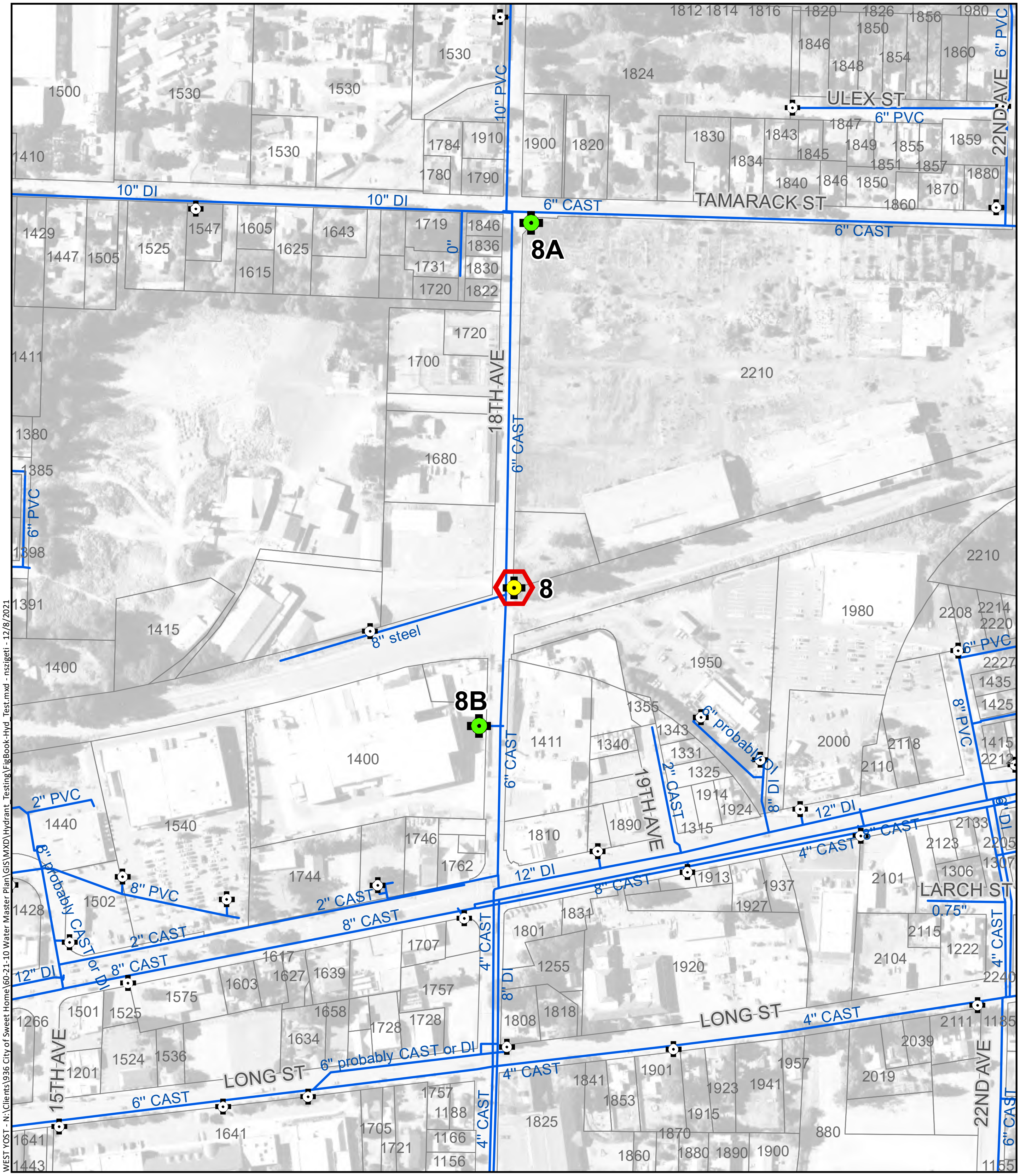
Scale in Feet

Figure A-7

Test 7

City of Sweet Home

Water Master Plan



WEST YOST - N:\Clients\936 City of Sweet Home\60-21-10 Water Master Plan\GIS\MXD\Hydrant_Testing\FigBook-Hyd_Test.mxd - nszigeti - 12/8/2021

Flowing Hydrant

Observation Hydrant

Hydrant

Pump Station

Storage Tank

Pipeline

N

0 150 300

Scale in Feet

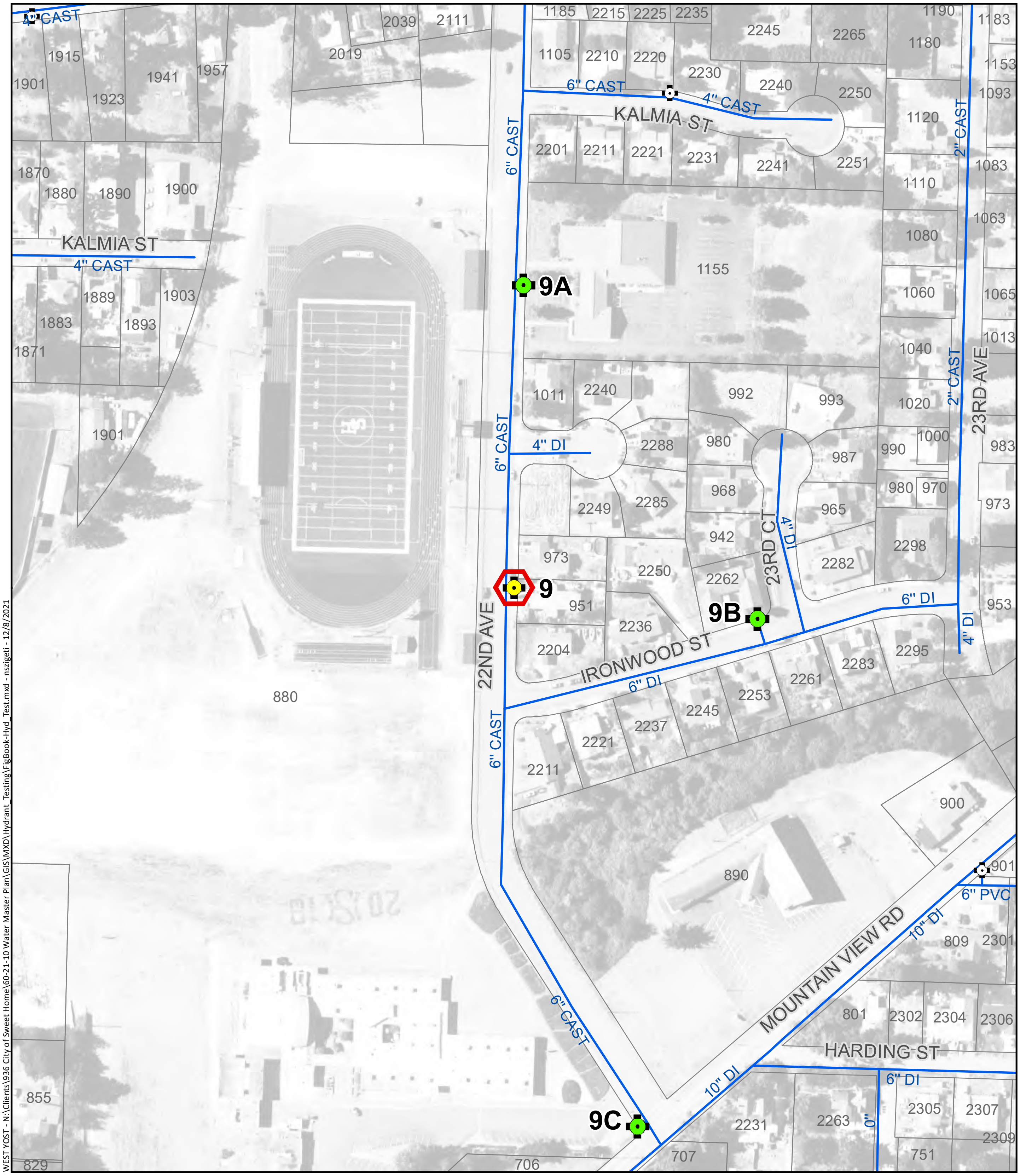
WEST YOST

Figure A-8

Test 8

City of Sweet Home

Water Master Plan



Flowing Hydrant
Observation Hydrant
Hydrant
Pump Station
Storage Tank
Pipeline

N

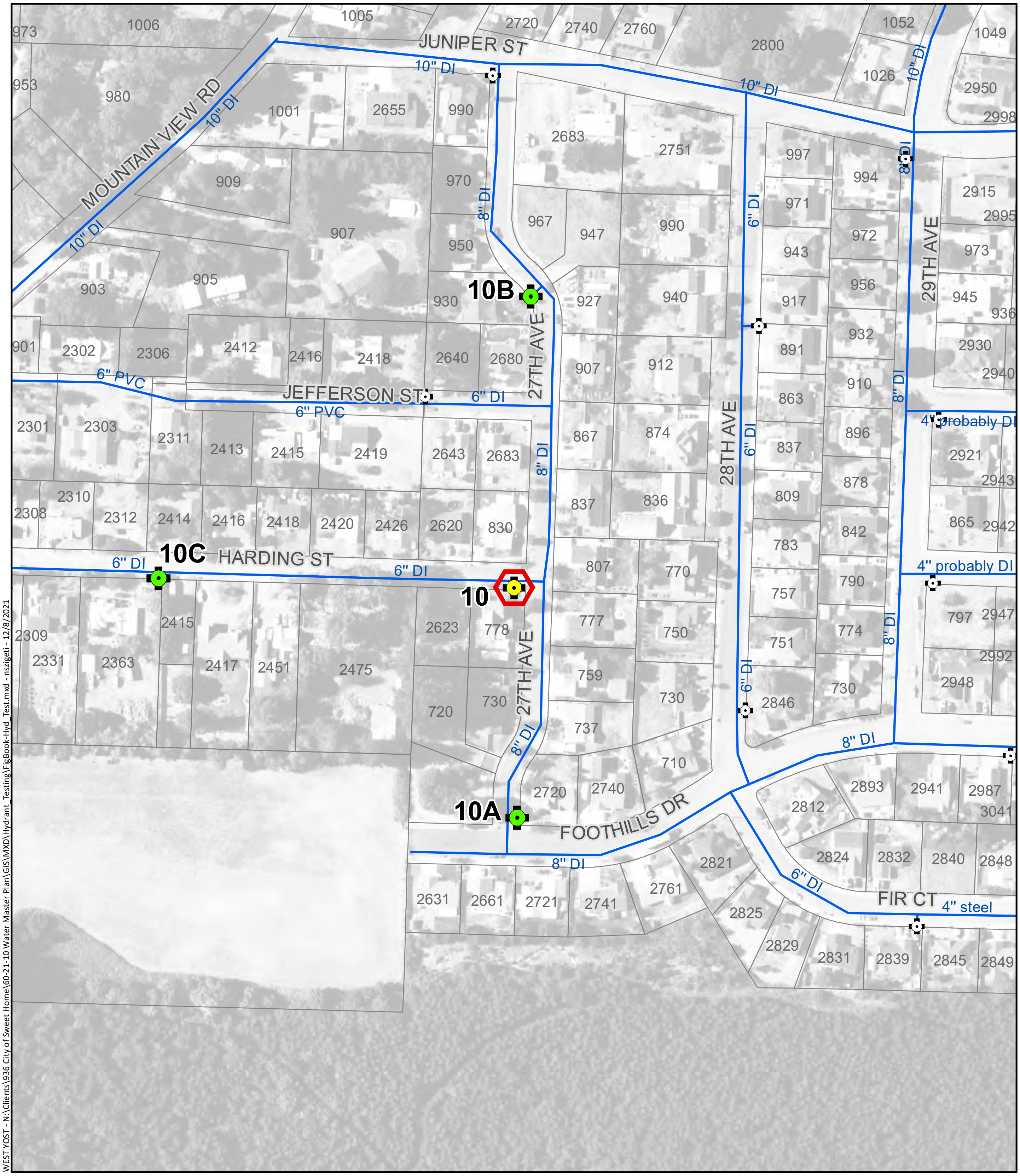
0 100 200
Scale in Feet

WEST YOST

Figure A-9

Test 9

City of Sweet Home
Water Master Plan



WEST YOST - N:\Clients\936 City of Sweet Home\60-21-10 Water Master Plan\GIS\MXD\Hydrant_Testing\FigBook-Hyd_Test.mxd - nszigeti - 12/8/2021

Flowing Hydrant
Observation Hydrant
Hydrant
Pump Station
Storage Tank
Pipeline

N

0 100 200
Scale in Feet

WEST YOST

Figure A-10

Test 10

City of Sweet Home
Water Master Plan

WEST YOST - N:\Clients\936 City of Sweet Home\60-21-10 Water Master Plan\GIS\MXD\Hydrant Testing\FigBook-Hyd_Test.mxd - nszigeti - 12/8/2021

Flowing Hydrant
Observation Hydrant
Hydrant
Pump Station
Storage Tank
Pipeline

N

0 100 200
Scale in Feet

WEST YOST

Figure A-11

Test 11

City of Sweet Home
Water Master Plan

Flowing Hydrant

Observation Hydrant

Hydrant

Pump Station

Storage Tank

Pipeline

N

0 100 200

Scale in Feet

WEST YOST

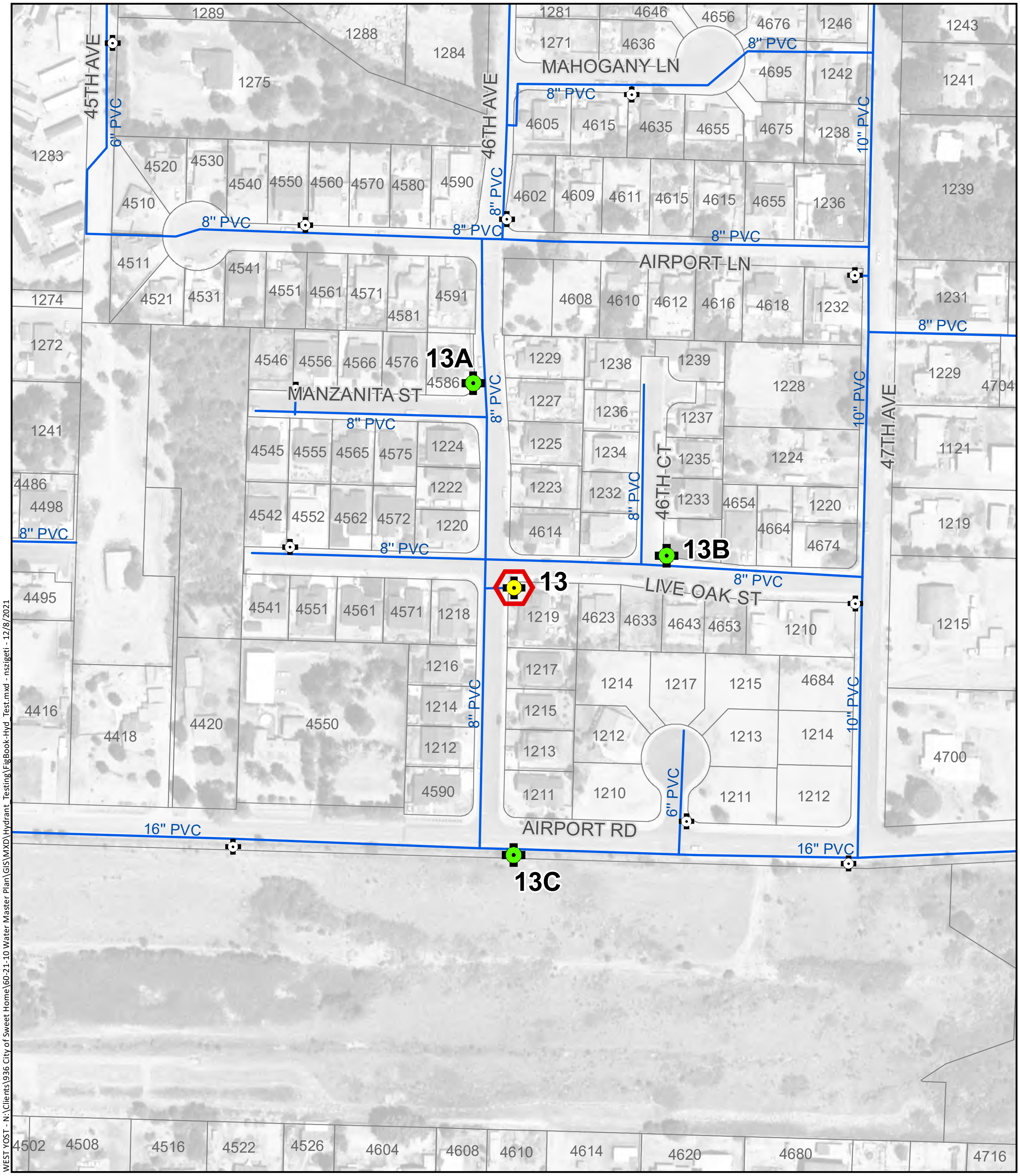
Figure A-12

Test 12

City of Sweet Home

Water Master Plan

WEST YOST - N:\Clients\936 City of Sweet Home\60-21-10 Water Master Plan\GIS\MXD\Hydrant_Testing\FigBook-Hyd_Test.mxd - nszigeti - 12/8/2021



Flowing Hydrant

Observation Hydrant

Hydrant

Pump Station

Storage Tank

Pipeline

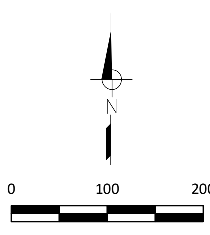
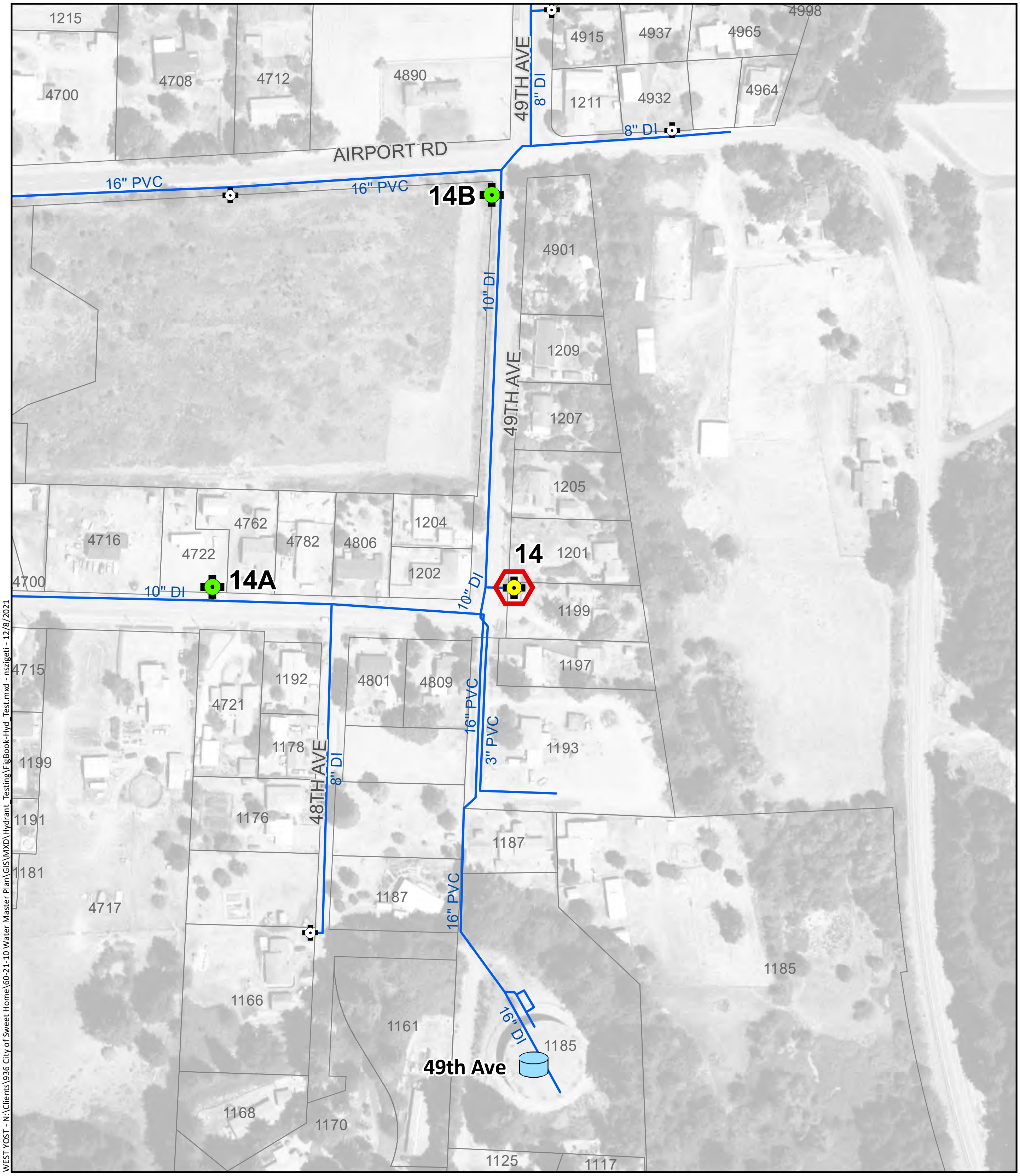


Figure A-13

Test 13

City of Sweet Home
Water Master Plan



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Flowing Hydrant
Observation Hydrant
Hydrant
Pump Station
Storage Tank
Pipeline

N

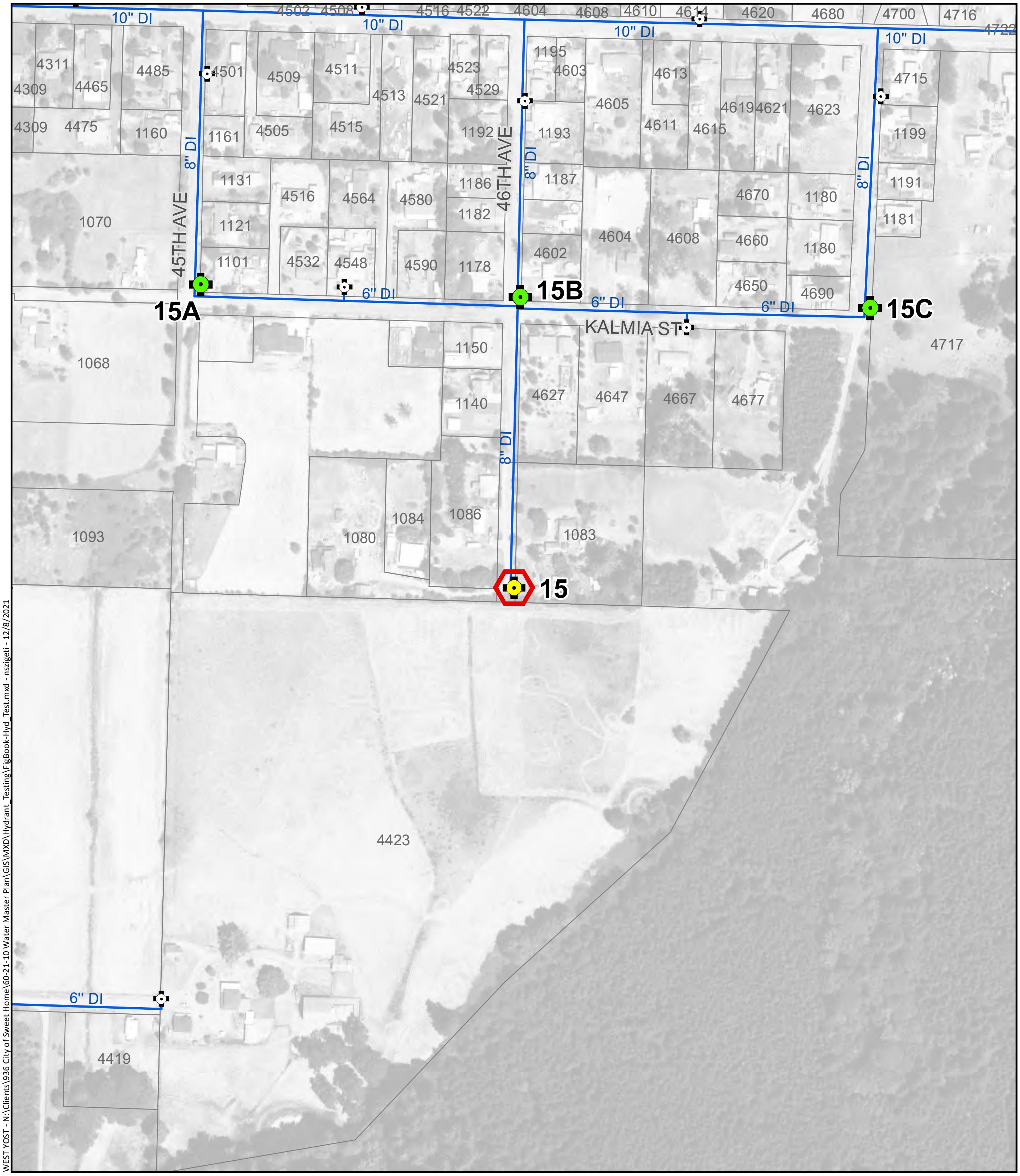
0 100 200
Scale in Feet

WEST YOST

Figure A-14

Test 14

City of Sweet Home
Water Master Plan



Flowing Hydrant

Observation Hydrant

Hydrant

Pump Station

Storage Tank

Pipeline

N

0 150 300

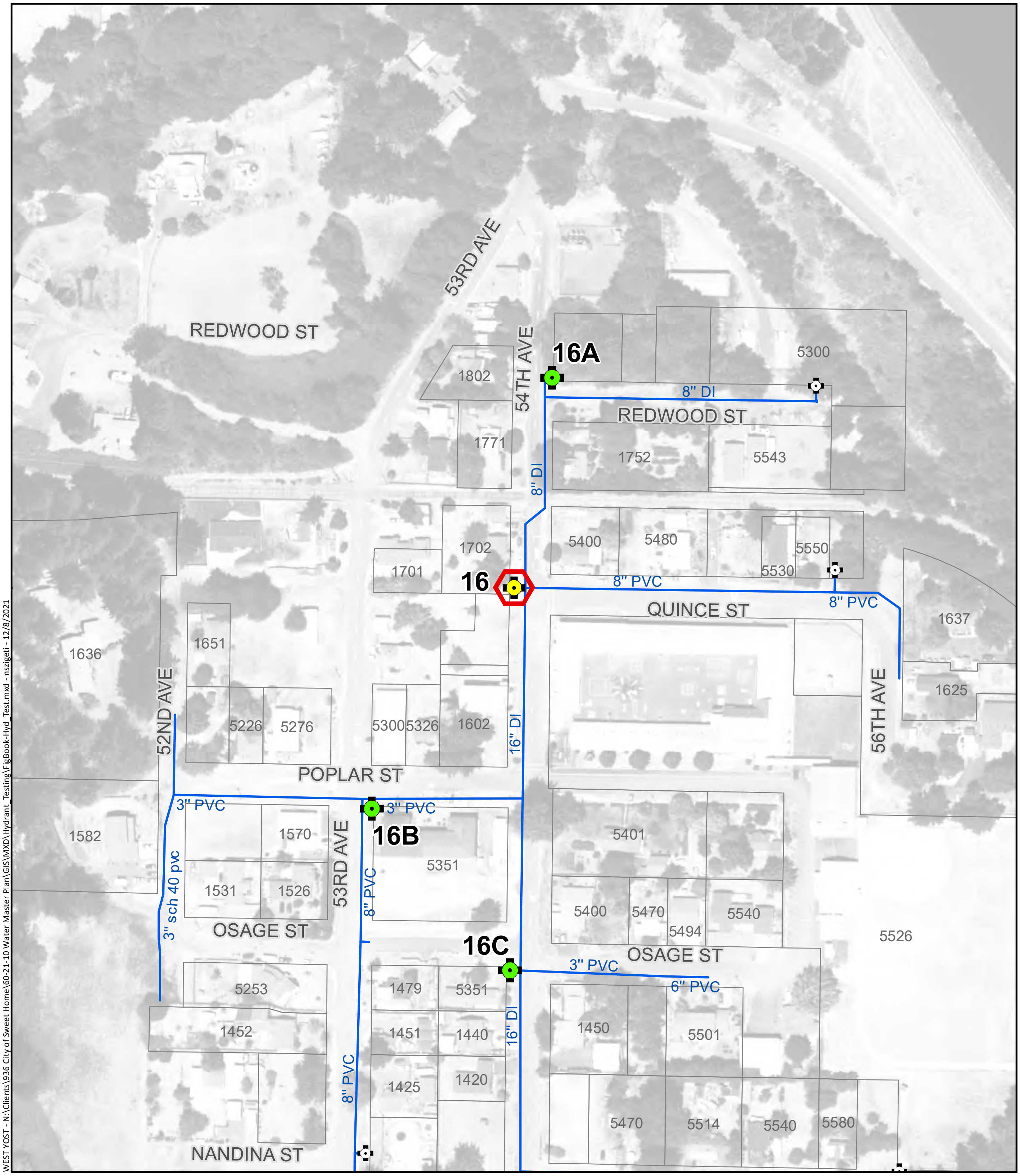
Scale in Feet

Figure A-15

Test 15

City of Sweet Home

Water Master Plan



Flowing Hydrant

Observation Hydrant

Hydrant

Pump Station

Storage Tank

Pipeline

N

0 100 200

Scale in Feet

WEST YOST

Figure A-16

Test 16

City of Sweet Home

Water Master Plan

Flowing Hydrant

Observation Hydrant

Hydrant

Pump Station

Storage Tank

Pipeline

N

0 250 500

Scale in Feet

WEST YOST

Figure A-17

Test 17

City of Sweet Home

Water Master Plan

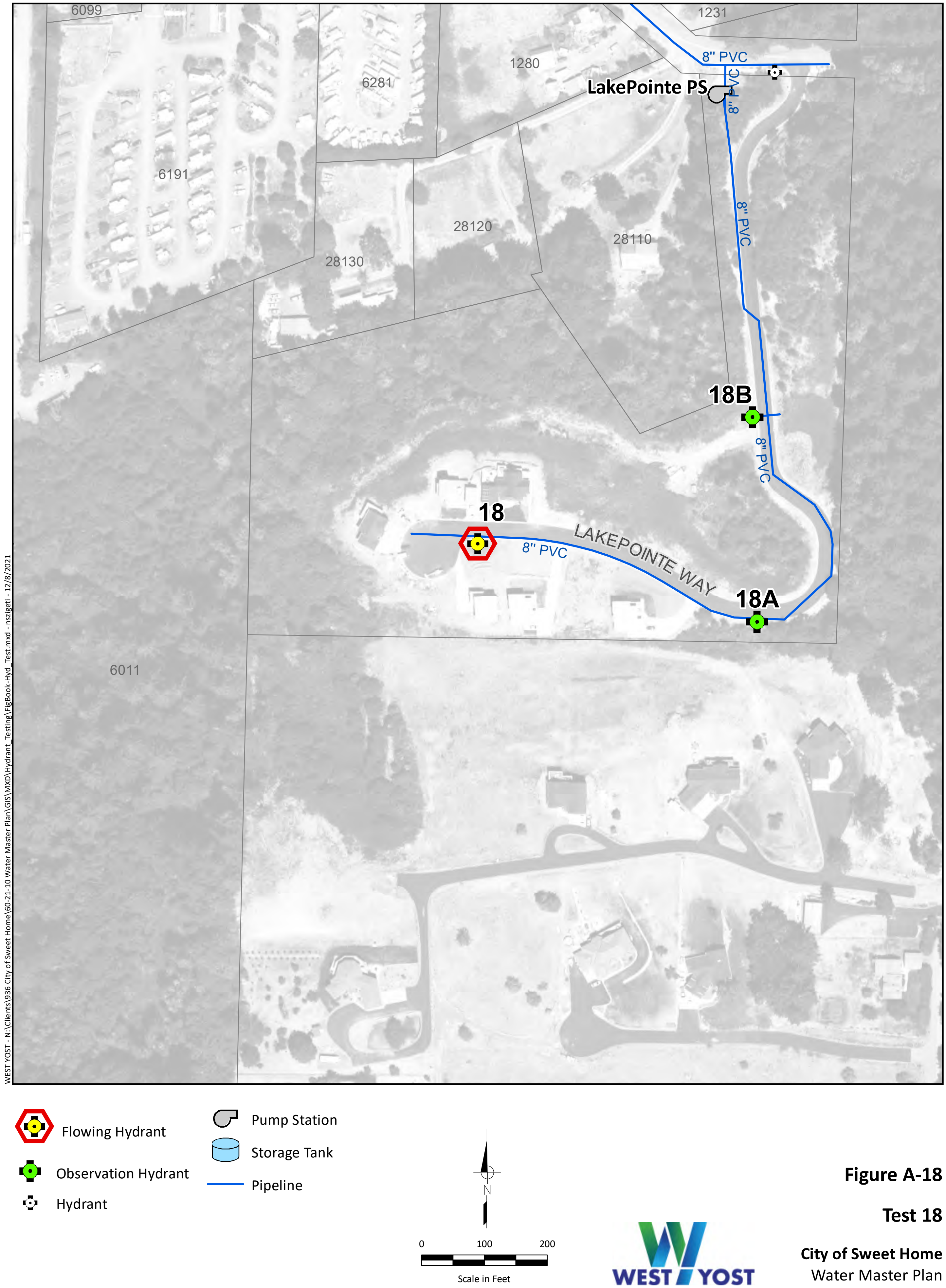


Figure A-18

Test 18

City of Sweet Home
Water Master Plan

Attachment B

Hydrant Test Data Logs

Table B-1. Data Log - Flowing Hydrant						
Hydrant Test No.	Date	Time Recorded	Hydrant Static Pressure, psi (note ±psi, if varies)	Hydrant Residual Pressure, psi (note ±psi, if varies)	Hydrant Flow, gpm (note ±gpm, if varies)	Comments / Notable Test Anomalies
1						
2						
3						
4						
5						
6						
7						
8						
9						
10						
11						
12						
13						
14						
15						
16						
17						
18						

Table B-2. Data Log - Monitoring Hydrant A

Hydrant Test No.	Monitoring Hydrant No.	Date	Time Recorded	Hydrant Static Pressure, psi (note ±psi, if varies)	Hydrant Residual Pressure, psi (note ±psi, if varies)	Comments / Notable Test Anomalies
1						
2						
3						
4						
5						
6						
7						
8						
9						
10						
11						
12						
13						
14						
15						
16						
17						
18						

Table B-2. Data Log - Monitoring Hydrant B						
Hydrant Test No.	Monitoring Hydrant No.	Date	Time Recorded	Hydrant Static Pressure, psi (note ±psi, if varies)	Hydrant Residual Pressure, psi (note ±psi, if varies)	Comments / Notable Test Anomalies
1						
2						
3						
4						
5						
6						
7						
8						
9						
10						
11						
12						
13						
14						
15						
16						
17						
18						

Table B-2. Data Log - Monitoring Hydrant C

Hydrant Test No.	Monitoring Hydrant No.	Date	Time Recorded	Hydrant Static Pressure, psi (note ±psi, if varies)	Hydrant Residual Pressure, psi (note ±psi, if varies)	Comments / Notable Test Anomalies
1						
2						
3						
4						
5						
6						
7						
8						
9						
10						
11						
12						
13						
14						
15						
16						
17						
18						

Appendix B

Geotechnical Seismic Risks and Hazards Mapping

Technical Memorandum			
To:	Sandrine Ganry West Yost	Project:	Sweet Home Water Master Plan
From:	Wolfe Lang, PE Delve Underground	cc:	
Prepared by:	Luke Ferguson, PE Delve Underground	Job No.:	6342.0
Date:	May 31, 2023		
Subject:	Seismic Hazards Evaluation - FINAL		

1.0 Introduction

The City of Sweet Home (City) is currently conducting a seismic resiliency study for their water system. A key required component of the study is understanding the seismic hazards present in the service area. The City has contracted West Yost to provide professional services for the resiliency study. West Yost has retained Delve Underground to conduct a seismic hazards assessment. The primary purpose of this task is to broadly identify the seismic hazard potentials, namely the strong ground shaking potential and seismic permanent ground deformation (PGD) in the Sweet Home service area. This task includes creating seismic hazard maps.

This memorandum presents the results of our evaluation. The following tasks were completed in accordance with our scope of work:

1. Review of available local geologic information;
2. Review of DOGAMI seismic hazard maps for a magnitude 9.0 Cascadia Subduction Zone (CSZ) event;
3. Review of available geotechnical boring and well log information to verify DOGAMI seismic hazard maps;
4. Development of estimates of seismic hazards in the project area, including strong ground shaking, liquefaction-induced settlement, lateral spreading displacement, and seismic landslide slope instability.
5. Development of hazard maps illustrating these hazards in relation to the Sweet Home service area;
6. Development of site response spectral acceleration values for a maximum considered earthquake (MCE_R) and a CSZ seismic event;
7. Development of this memorandum summarizing the results of our evaluations.

2.0 Data Review

Delve Underground performed a background information review and reviewed available existing geotechnical data from various previous projects within the Sweet Home service area. Existing geotechnical data sources consisted mainly of well logs. Limited subsurface information was provided by the City at the 49th Ave Reservoir and the Strawberry Reservoir.

3.0 Geologic and Seismic Setting

3.1 Geologic Setting

The Sweet Home service area is located in the foothills of the Western Cascades, a north-south trending physiographic region that stretches from northern California to British Columbia, tucked between the Willamette Valley to the west and the younger High Cascades to the east. The Western Cascades in Oregon were formed by a series of volcanic events from approximately 35 to 17 million years ago. The region is marked by densely forested hills dissected by the region's many rivers (Madin, 1990; Schlicker and Deacon, 1967; Wilson, 1998; Popowski, 1996).

The Paleogene structural basement of this region of the Western Cascades is composed of non-marine volcaniclastic sedimentary rocks, tuff, basaltic andesite, andesite, and dacite of the Late Eocene to Oligocene Fisher Formation. The Fisher Formation is overlain by basalt lavas, ash-flow tuff, tuff, and non-marine sedimentary rocks of the Little Butte Volcanic Series. A subducting plate below the Eocene shoreline resulted in a volcanic chain that produced the volcanic activity responsible for the Fisher Formation and the Little Butte Volcanic Series. As the angle of the subducting plate shifted, the volcanic activity gradually shifted east of the region.

Over the span of geologic time, Quaternary sedimentary deposits of alluvium, colluvium, landslide deposits, and terrace deposits have accumulated on the volcanic rock surfaces and in the valleys formed by the rivers. The sediments consist primarily of unconsolidated gravel and sand, with lenses of silt and clay.

3.2 Seismic Setting

The Pacific Northwest is located near an active tectonic plate boundary. Off the northwest coast the Juan de Fuca oceanic plate is subducting beneath the North American crustal plate. This tectonic regime has resulted in seismicity in the project area occurring from three primary sources:

- Shallow crustal faults within the North American plate;
- CSZ intraplate faults within the subducting Juan de Fuca plate; and
- CSZ megathrust events generated along the boundary between the subducting Juan de Fuca plate and the overriding North American plate.

Among these three sources, CSZ megathrust events are considered as having the most hazard potential due to the anticipated magnitude and duration of associated ground shaking. Recent studies indicate that the CSZ can potentially generate large earthquakes with magnitudes ranging from 8.0 to 9.2 depending on rupture length. The recurrence intervals for CSZ events are estimated at approximately 500 years for the mega-magnitude full rupture events (magnitude 9.0 to 9.2) and 200 to 300 years for the large-magnitude partial rupture events (magnitude 8.0 to 8.5). Additionally, current research indicates a probability of future occurrence because the region is "past due" based on historic and prehistoric recurrence intervals documented in ocean sediments. For example, over the next 50 years, the CSZ earthquake has an estimated probability of occurrence off the Oregon Coast on the order of 16 to 22 percent (Goldfinger et. al., 2016).

4.0 Subsurface Conditions

The subsurface within the project area is dominated by the following geologic units:

- **Alluvium**: Consists of unconsolidated gravel, sand, silt, and clay deposited along active stream channels and their adjoining flood plains and is Holocene in age.
- **Colluvium**: Consists of an unconsolidated mixture of soil and rock fragments that have been transported downslope by precipitation and gravity via surficial erosion. This unit is present mainly on and at the base of steep slopes.
- **Landslide Deposits**: Consists of unconsolidated mixed masses of rock and soil deposited by gravity-driven mass-wasting processes such as slumps, landslides, debris flows, etc. Individual slide masses can form large complexes resulting from long-term landslide activity.
- **Mixed Grain Sediments**: Consists primarily of unconsolidated deposits of gravel and sand, with some silt and clay, and is considered to be Pleistocene-aged based on stratigraphy.
- **Coarse Grained Sediments**: Consists primarily of gravel with minor sand and silt deposited by steeper gradient streams draining the Western Cascades. This unit is assigned a Holocene age based on location near active stream channels.
- **Sedimentary Rock**: Consists primarily of Tertiary-aged sandstones and conglomerates, including sedimentary rock units of volcaniclastic origin.
- **Volcanic Rock**: Consists primarily of Tertiary-aged basalt and diabase associated with Western Cascade and Little Butte volcanic activity.

A geology map of the Sweet Home service area is shown in Figure 1.

5.0 Geotechnical Seismic Hazards

Seismic hazards include strong ground shaking, liquefaction settlement, lateral spreading, and seismic-induced landslides. These hazards have the potential to damage facilities (i.e., treatment plant, pipelines, reservoirs, pump stations) through either permanent ground deformation or intense shaking. Our analysis of these seismic hazards is based on information provided from existing geotechnical explorations, historic well logs, DOGAMI hazard maps created for the Oregon Resilience Plan (ORP) (Madin and Burns, 2013), and our knowledge of the geotechnical conditions of the area. In our seismic analyses we assumed a magnitude 9.0 earthquake and a bedrock peak ground acceleration of 0.13 g to represent the effects of a CSZ seismic event in the project area.

Geotechnical information contained in logs and reports studied for this project was analyzed for potential seismic hazards and compared to seismic hazards mapped by DOGAMI. Where appropriate, DOGAMI mapped hazards were modified and improved to incorporate results of the analysis of local geotechnical information. Of note, existing geotechnical information in the project area is sparse, with quality subsurface information available mainly only at reservoir, water treatment, and wastewater treatment sites. Subsurface conditions could not be confirmed where subsurface investigations are not available.

5.1 Ground Shaking (Peak Ground Velocity)

To assess the hazard potential of ground shaking in the project area we reviewed the peak ground velocity (PGV) map published by DOGAMI for the ORP in the event of a M9 CSZ earthquake (Madin and Burns, 2013).

The estimated ground shaking intensity (PGV) depends on earthquake magnitude, distance to fault rupture, and the subsurface materials present at the site. Generally, in the Sweet Home service area the PGV values are estimated to range between 5 and 10 inches per second. The PGV hazard map for the Sweet Home service area is shown in Figure 2.

5.2 Liquefaction

Liquefaction is a phenomenon affecting saturated, granular soils in which cyclic, rapid shearing from an earthquake results in a drastic loss of shear strength and a transformation from a granular solid mass to a viscous, heavy fluid mass. The results of soil liquefaction include loss of shear strength, loss of soil materials through sand boils, flotation of buried chambers/pipes, and post liquefaction settlement.

To evaluate the hazard potential of soil liquefaction in the project area, we reviewed liquefaction hazard maps published by DOGAMI for the ORP, modified as discussed in Section 5.0, in the event of a M9 CSZ earthquake. Where geotechnical data was available, we conducted site specific analyses based on the subsurface conditions shown in previous geotechnical explorations using the latest SPT-based liquefaction susceptibility and settlement assessment procedures (Boulanger and Idriss, 2014; Idriss and Boulanger, 2008). Based on our evaluation, liquefaction is not a significant hazard across the majority of the Sweet Home service area. Coarse gravels overlying shallow bedrock provide subsurface conditions that are not conducive to liquefaction. At the wastewater treatment plant existing geotechnical investigations show isolated pockets of unconsolidated fill soils that have the potential to liquefy. These fill pockets are discontinuous and not expected to present a significant hazard to existing water system facilities. The Sweet Home service area liquefaction hazard map is shown in Figure 3.

5.3 Lateral Spreading

Liquefaction can result in progressive horizontal deformation of the ground known as lateral spreading. The lateral movement of liquefied soil breaks the non-liquefied soil crust into blocks that progressively move downslope or toward a free face in response to earthquake generated ground accelerations. Seismic movement incrementally pushes these blocks downslope as seismic accelerations overcome the strength of the liquefied soil column. The potential for and magnitude of lateral spreading depends on the liquefaction potential of the soil, the magnitude and duration of earthquake ground accelerations, the site topography, and the post-liquefaction strength of the soil.

To assess the hazard potential of lateral spreading in the project area, we reviewed a lateral spreading hazard map published by DOGAMI for the ORP, modified as discussed in Section 5.0, in the event of a M9 CSZ earthquake. Based on our evaluation, lateral spreading is not expected to be a hazard in the Sweet Home service area. Therefore, a lateral spreading hazard map is not included as part of this memorandum.

5.4 Seismic Landslides

Earthquake induced landslides can occur on slopes due to the inertial force from an earthquake adding load to a slope. The ground movement due to landslides can be extremely large and damaging to pipelines and other structures. To assess the hazard potential of landslides in the project area, we reviewed a landslide hazard map published by DOGAMI for the Sweet Home area, and modified it based on reviewed geotechnical data, site topography, and the location of mapped historic and prehistoric landslide deposits.

Generally, the seismic landslide hazard for the study area is low due to its relative flatness. However, seismic landslide hazard is present in isolated areas where steeper slopes are present along the southern boundary of the service area. Specifically, there is a potential for seismic landslides at steep slopes adjacent to the 10th Avenue and 49th Avenue reservoirs. Seismic landslide PGD up to 4 feet may occur in these areas. The seismic landslide hazard map of the service area is shown in Figure 5, with the hazard quantified by estimated seismic landslide induced PGD. Mapped existing landslide deposits are also shown.

6.0 Spectral Accelerations

Seismic spectral acceleration parameters for PGA_M, S_{M1}, and S_{MS} were estimated for the project area by Delve Underground for both a MCE_R and a CSZ earthquake. The MCE_R roughly corresponds to a seismic event with a 2,475-year recurrence interval and the CSZ roughly corresponds to a seismic event with a 475-year recurrence interval.

Spectral accelerations for the MCE_R event were determined in a probabilistic manner using the hazard tool published online by ASCE 7, which draws its spectral acceleration values from the ASCE 7-22 building code. A Risk Category of III was assumed for the Sweet Home water system.

Spectral accelerations for the CSZ event were determined in a deterministic manner using the NGA-Subduction Ground Motion Characterization Tool (Mazzoni, 2020) in conjunction with the online United States Geologic Survey (USGS) Unified Hazard Tool. This tool provides a range of estimated spectral accelerations based on the magnitude and rupture distance of a specific earthquake event. A magnitude of 9.0 and a rupture distance of 87 km were assumed. The 50th percentile values are presented in this study.

These spectral acceleration parameters are dependent on the seismic site class of the soil at the site. To assess the seismic site classes present in the project area, we reviewed a site class map published by DOGAMI for the Sweet Home area, and modified it based on reviewed geotechnical data. Estimated spectral accelerations for a CSZ event are shown in Figure 5 and estimated spectral accelerations in an MCE_R event are provided in Figure 6. These values are also presented in Table 1.

Table 1. Spectral Accelerations

Site Class	CSZ Event			MCE_R Event		
	PGA_M (g)	S_{M1} (g)	S_{MS} (g)	PGA_M (g)	S_{M1} (g)	S_{MS} (g)
B	0.14	0.14	0.27	0.30	0.28	0.58
C	0.21	0.23	0.40	0.38	0.44	0.83
D	0.27	0.50	0.38	0.42	0.69	0.95

7.0 Conclusions

The majority of the Sweet Home service area is not located within a seismic hazard zone. The subsurface is dominated by coarse gravels and shallow bedrock, without significant deposits of liquefiable soils. Therefore, the liquefaction and lateral spreading hazard in the service area is low. Certain areas of unconsolidated fill materials, such as those present at the wastewater treatment plant, are liquefiable. However, these fill materials are discontinuous and not expected to pose a significant hazard to the Sweet Home water system. It is important to note that available subsurface information in the service area is limited and subsurface conditions could not be confirmed where existing geotechnical information was not available.

There is a seismic landslide hazard present on slopes along the southern boundary of the service area, including at the 10th Avenue and 49th Avenue reservoir sites. Delve Underground recommends that site specific slope stability analyses, including additional subsurface investigations, be performed at both the 10th Avenue and 49th Avenue reservoirs to determine the level of seismic landslide hazard present at those sites.

8.0 Limitations

This Seismic Hazards Technical Memorandum has been prepared for the Sweet Home Water Master Plan project, located in Sweet Home, Linn County, Oregon. This report contains a compilation of information from previous studies, projects, and published literature. The professional judgements and characterizations presented herein are based on this information. Delve Underground is not responsible for errors and omissions that might appear in studies reported by others.

The scope of our geotechnical services has not included an environmental evaluation regarding the presence or absence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below the site.

This report has been completed within the limitations of the West Yost Associates, Inc. approved scope of work, schedule, and budget. The services rendered have been performed in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions in the same area. Delve Underground is not responsible for the use of this report for anything other than the Sweet Home Water Master Plan project.

DELVE UNDERGROUND

Luke Ferguson, P.E.
Project Engineer

Yuxin “Wolfe Lang”, P.E., G.E.
Principal Engineer

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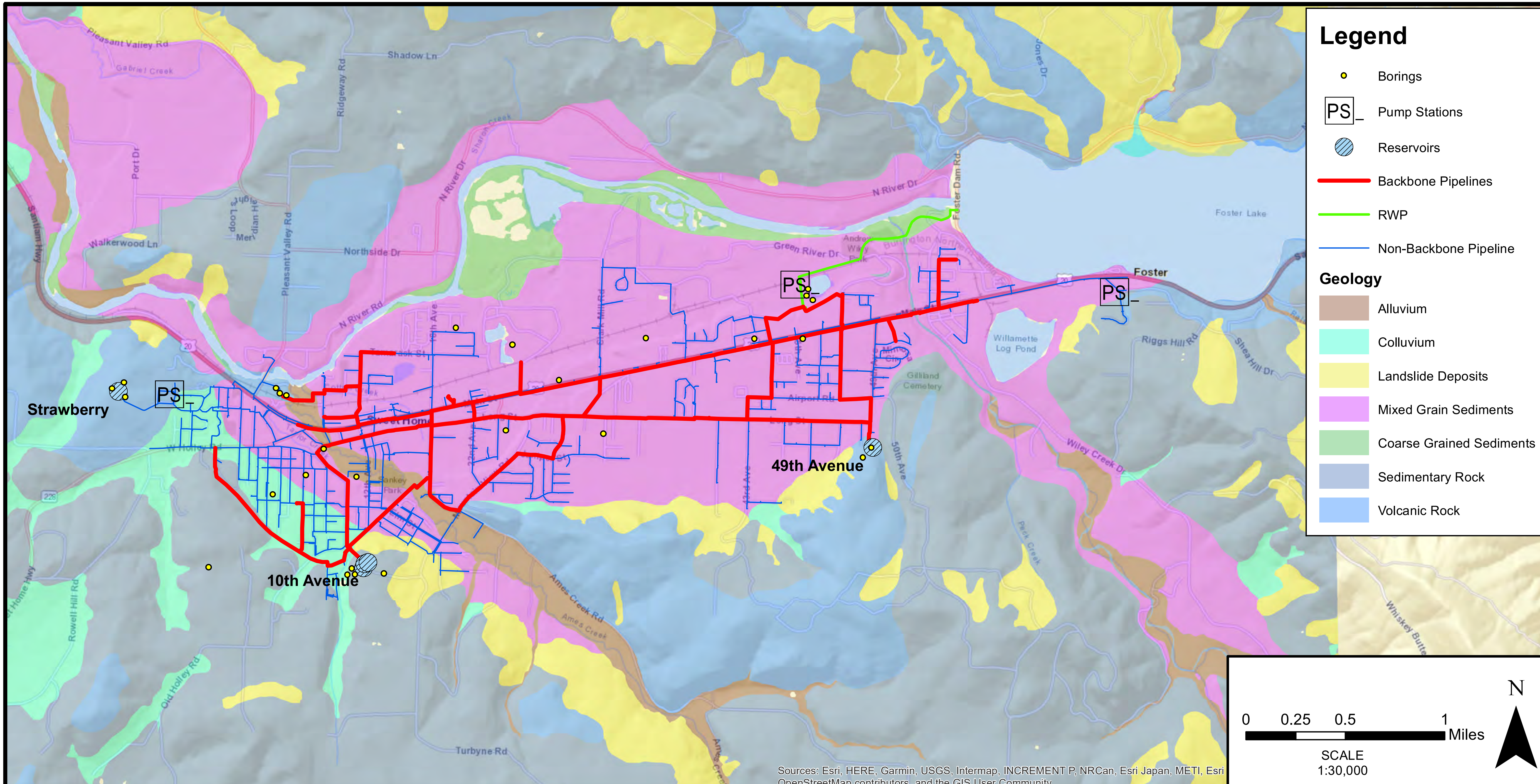
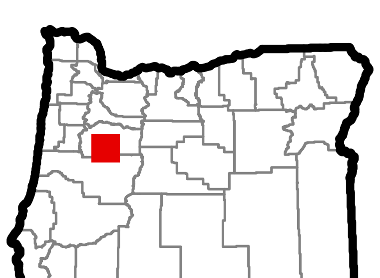
Figures





DELVE
underground

CITY OF SWEET HOME WATER SYSTEM EVALUATION
LINN COUNTY, OREGON

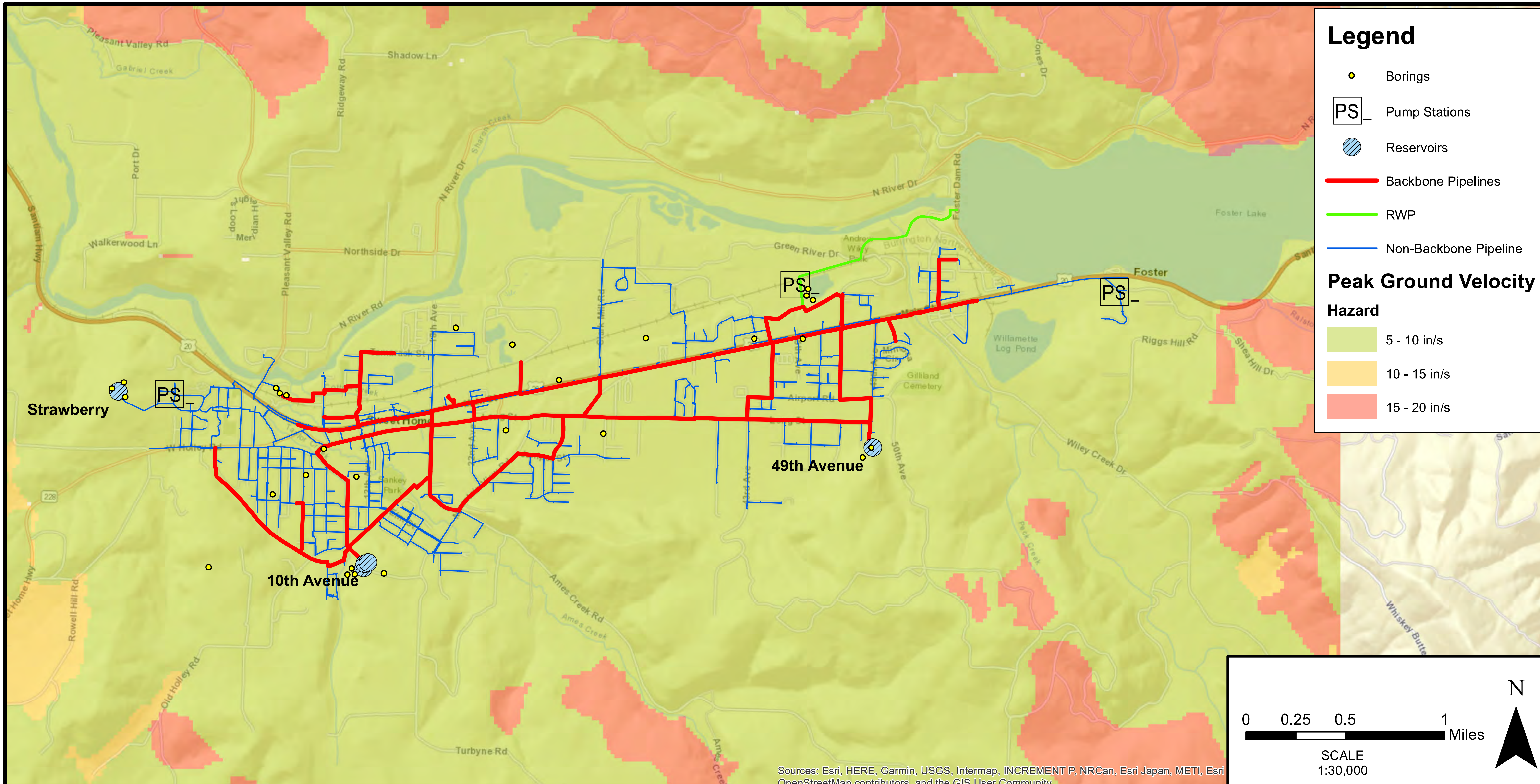
SEISMIC HAZARDS TECHNICAL MEMORANDUM
GEOLOGY MAP

MAY
2023

FIGURE
1

NOTES:
GEOLOGY MAP ESTIMATES SHOWN ARE BASED ON DATA FROM EXISTING BORINGS AND DOGAMI OPEN FILE REPORT O-13-06.

AREAS OUTSIDE OF EXISTING BORINGS HAVE NOT BEEN VERIFIED.



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CITY OF SWEET HOME WATER SYSTEM EVALUATION
LINN COUNTY, OREGON

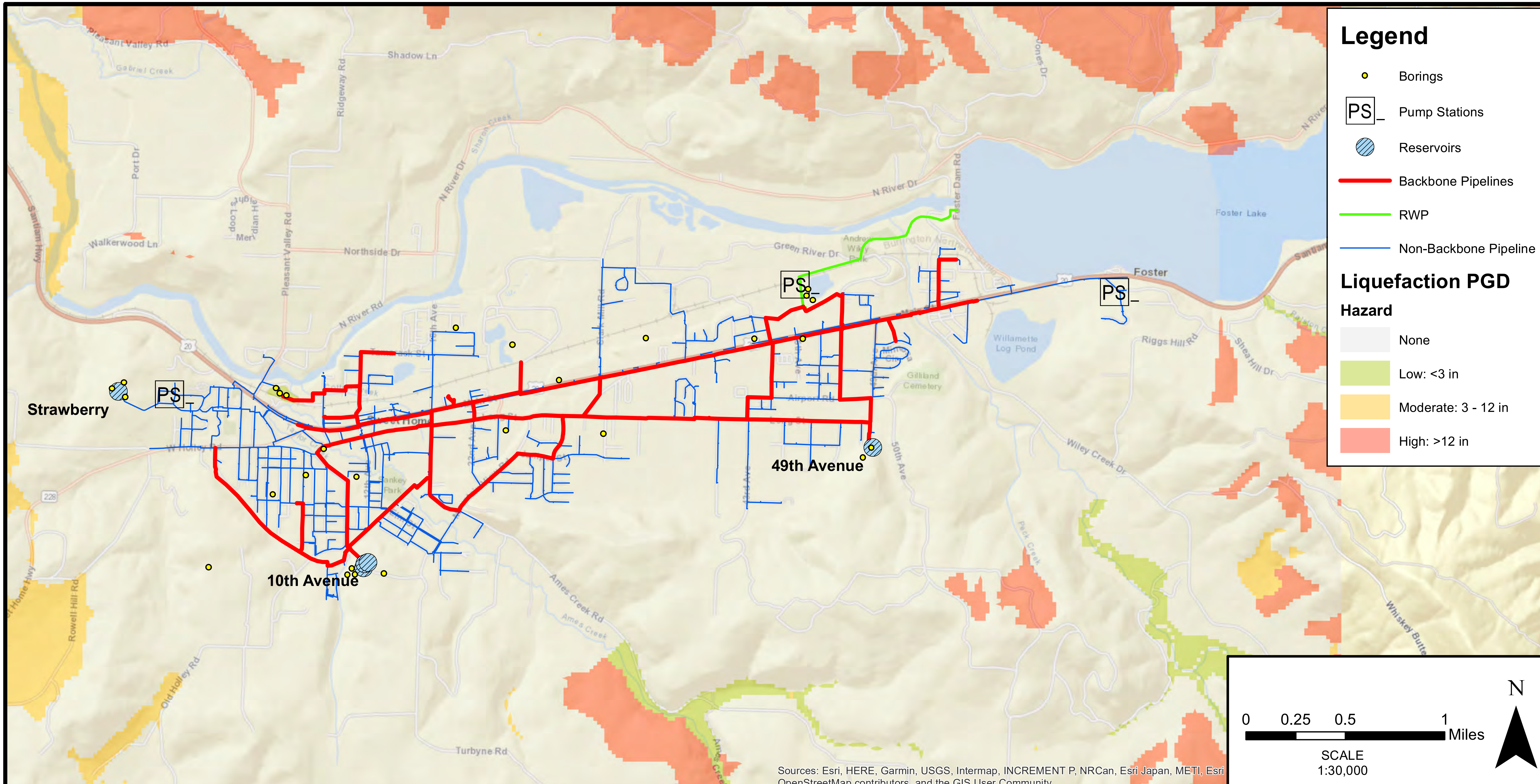
SEISMIC HAZARDS TECHNICAL MEMORANDUM
PEAK GROUND VELOCITY MAP

MAY 2023

FIGURE 2

NOTES:
PEAK GROUND VELOCITY MAP ESTIMATES SHOWN ARE BASED ON DATA FROM EXISTING BORINGS AND DOGAMI OPEN FILE REPORT O-13-06.

AREAS OUTSIDE OF EXISTING BORINGS HAVE NOT BEEN VERIFIED.



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CITY OF SWEET HOME WATER SYSTEM EVALUATION
LINN COUNTY, OREGON

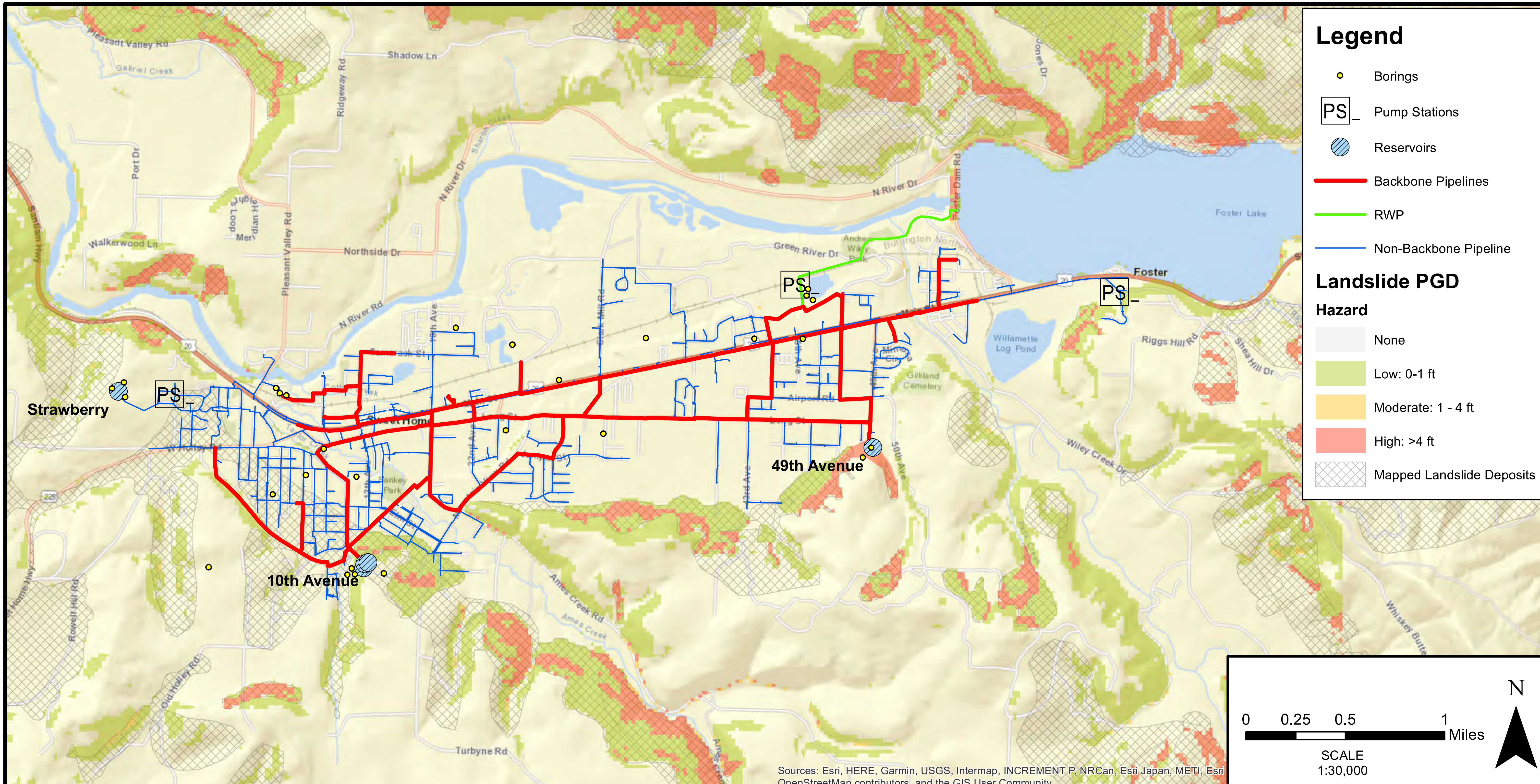
SEISMIC HAZARDS TECHNICAL MEMORANDUM
LIQUEFACTION SETTLEMENT MAP

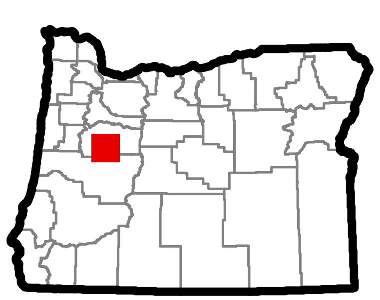
MAY 2023

FIGURE 3

NOTES:
LIQUEFACTION SETTLEMENT MAP ESTIMATES SHOWN ARE BASED ON DATA FROM EXISTING BORINGS AND DOGAMI OPEN FILE REPORT O-13-06 AND SITE OBSERVATIONS.

AREAS OUTSIDE OF EXISTING BORINGS HAVE NOT BEEN VERIFIED.





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CITY OF SWEET HOME WATER SYSTEM EVALUATION
LINN COUNTY, OREGON

SEISMIC HAZARDS TECHNICAL MEMORANDUM
SEISMIC LANDSLIDE MAP

MAY
2023

FIGURE
4

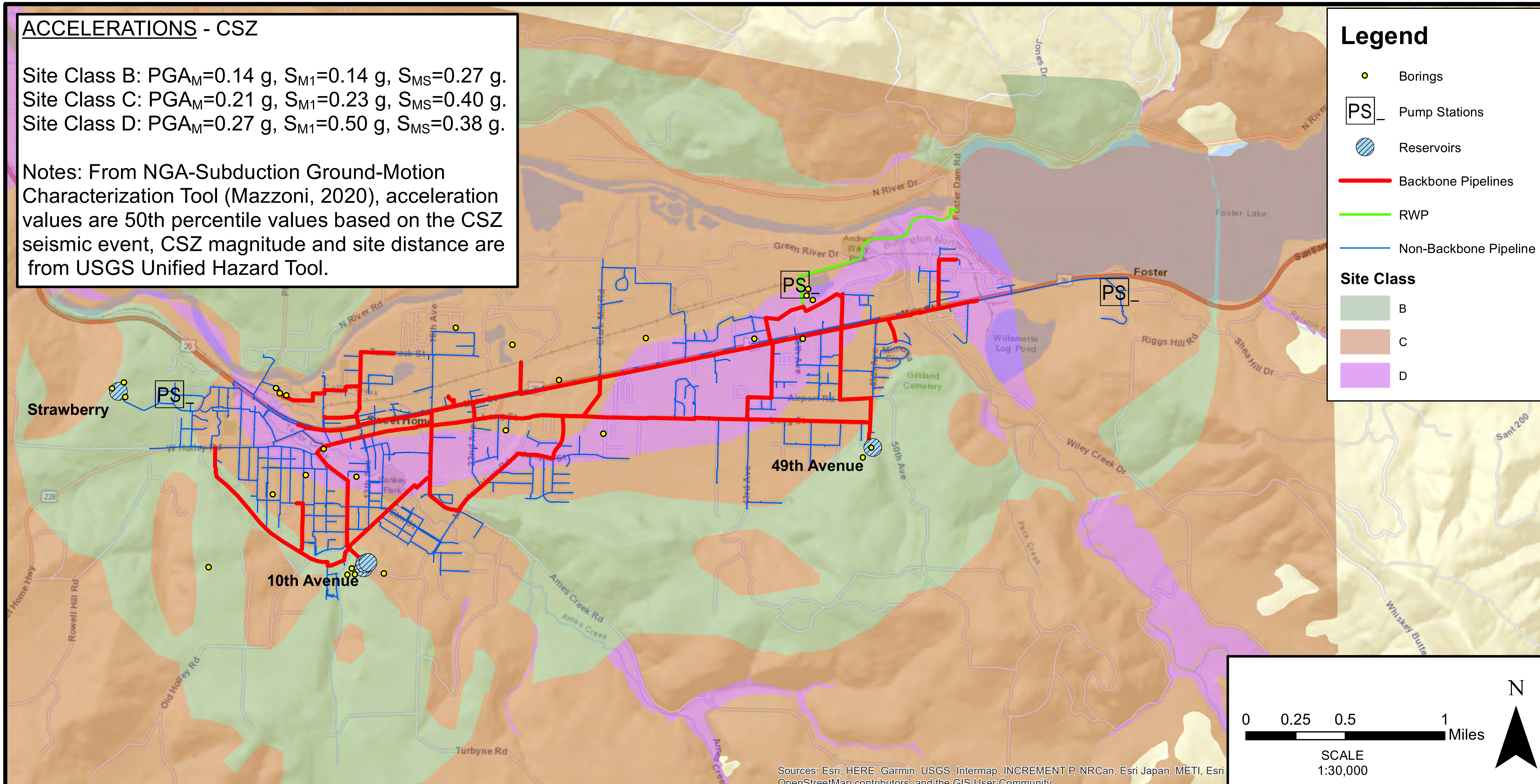
NOTES:
SEISMIC LANDSLIDE MAP ESTIMATES SHOWN ARE BASED ON DATA FROM EXISTING BORINGS AND DOGAMI OPEN FILE REPORT O-13-06 AND SITE OBSERVATIONS.

AREAS OUTSIDE OF EXISTING BORINGS HAVE NOT BEEN VERIFIED.

ACCELERATIONS - CSZ

Site Class B: PGA_M=0.14 g, S_{M1}=0.14 g, S_{MS}=0.27 g.
Site Class C: PGA_M=0.21 g, S_{M1}=0.23 g, S_{MS}=0.40 g.
Site Class D: PGA_M=0.27 g, S_{M1}=0.50 g, S_{MS}=0.38 g.

Notes: From NGA-Subduction Ground-Motion Characterization Tool (Mazzoni, 2020), acceleration values are 50th percentile values based on the CSZ seismic event, CSZ magnitude and site distance are from USGS Unified Hazard Tool.



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CITY OF SWEET HOME WATER SYSTEM EVALUATION
LINN COUNTY, OREGON

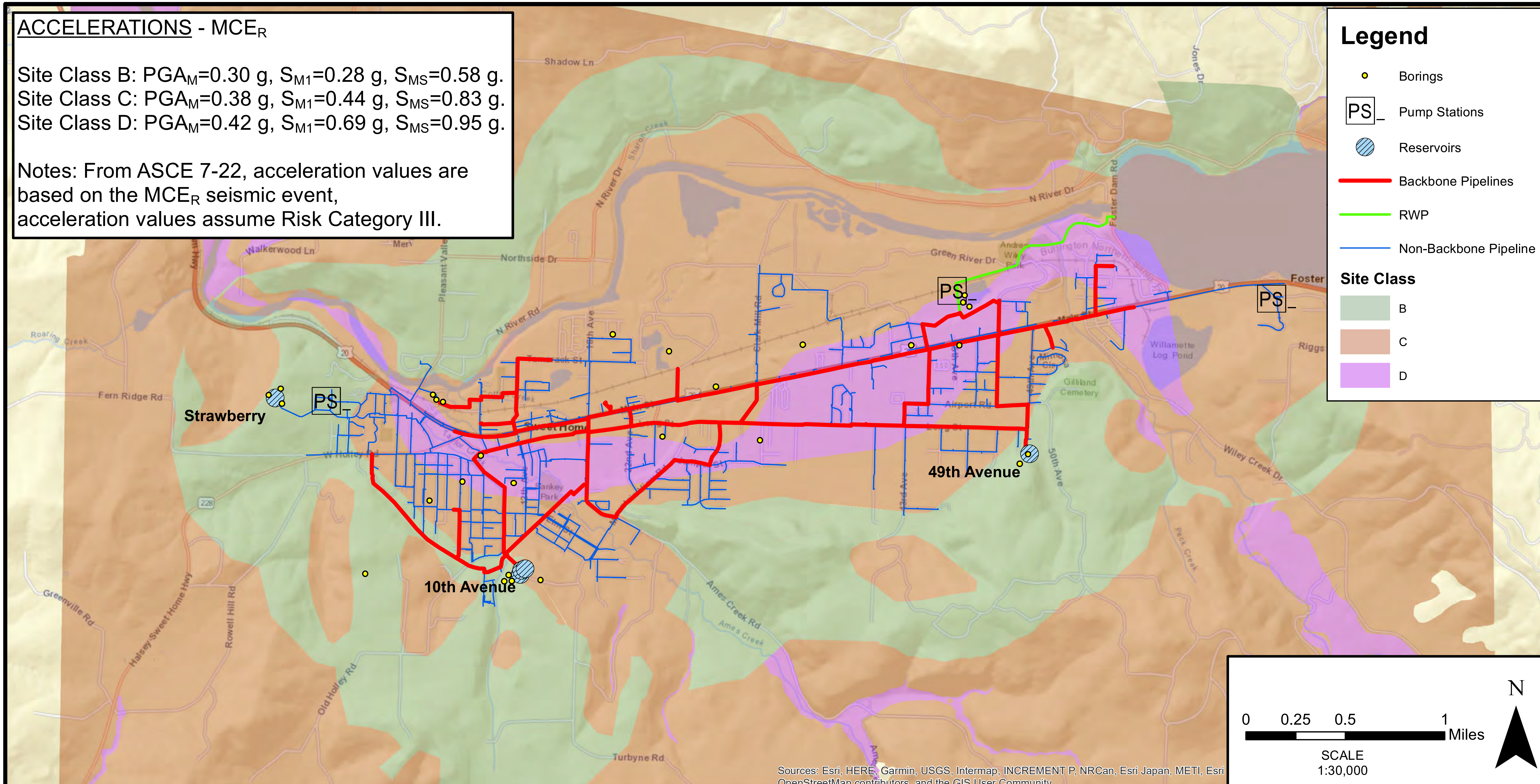
SEISMIC HAZARDS TECHNICAL MEMORANDUM
SPECTRAL ACCELERATIONS - CSZ

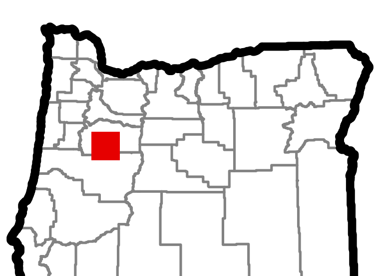
MAY 2023

FIGURE 5

NOTES:
SPECTRAL ACCELERATIONS - CSZ ESTIMATES SHOWN ARE BASED ON DATA FROM EXISTING BORINGS AND DOGAMI OPEN FILE REPORT O-13-06, USGS UHS, AND MAZZONI, 2020.

AREAS OUTSIDE OF EXISTING BORINGS HAVE NOT BEEN VERIFIED.





DELVE underground

CITY OF SWEET HOME WATER SYSTEM EVALUATION
LINN COUNTY, OREGON

SEISMIC HAZARDS TECHNICAL MEMORANDUM
SPECTRAL ACCELERATIONS - MCE_R

MAY 2023

FIGURE 6

NOTES:
SPECTRAL ACCELERATIONS - MCE_R ESTIMATES SHOWN ARE BASED ON DATA FROM EXISTING BORINGS AND DOGAMI OPEN FILE REPORT O-13-06 AND ASCE 7-22.

AREAS OUTSIDE OF EXISTING BORINGS HAVE NOT BEEN VERIFIED.

Appendix C

Structural Seismic Resiliency Evaluation

ACE ENGINEERING LLC
professional structural engineering
commercial . residential . industrial

po box 231 . ashland . oregon 97520
541.552.1417 . ace-engineeringllc.com

STRUCTURAL
TECHNICAL MEMORANDUM

DATE: **June 1, 2023**

TO: **WEST YOST**

ATTENTION: **SANDRINE GANRY**

PROJECT: **2021-33, CITY OF SWEET HOME, OREGON, WATER MASTER PLAN**

SUBJECT: **ASCE/SEI 41-17 SEISMIC EVALUATION OF EXISTING STRUCTURES**

1.0 Introduction

The City of Sweet Home, Oregon (City) is currently conducting a Water Master Plan (WMP) for their water treatment and distribution system. The City has retained West Yost to perform the WMP. West Yost retained ACE Engineering LLC to perform the structural portion of the WMP.

The primary purpose of the structural portion of the WMP is to broadly identify the potential structural and seismic deficiencies of each significant structure in the water treatment and distribution system. This memorandum presents the results of the structural evaluation. The following tasks were completed as the structural scope of work:

1. Review existing documentation of each structure that was made available by the City.
2. Review Seismic Hazards Evaluation prepared by McMillen Jacobs Associates, April 27, 2022.
3. Site observation of each significant structure in the water treatment and distribution system on June 13 and 14, 2022.
4. Abbreviated description of the structural system of each significant structure in the water treatment and distribution system.
5. Complete ASCE/SEI 41-17 Tier 1 Checklists, Quick Checks, and Evaluations.
6. Abbreviated summary of findings and identification of shortcomings of each significant structure in the water treatment and distribution system.

2.0 Documentation Review

The City provided original design drawings for each of the significant structures in the water treatment and distribution system. The drawings include:

1. Raw Water Intake (2007).
2. Raw Water Pump Station (2008)
3. Water Treatment Building (2008)
4. Water Treatment Pond (2008)
5. Lake Pointe Pump Station (2016)
6. Strawberry Pump Station (2001)
7. Strawberry Reservoir (2001)
8. Strawberry Reservoir Vault (2001)
9. 10th Avenue Reservoir 300k Inactive (1938)
10. 10th Avenue Reservoir 700k (1951)
11. 10th Avenue Reservoir 1.5M (1969)
12. 49th Avenue Reservoir (1993)

A review of the structural drawings and details that were provided by the City was performed. The Geotechnical engineers at McMillen Jacobs Associates provided their Technical Memorandum for Seismic Hazards Evaluation for each site occupied by the water distribution system. A review of the Seismic Hazards Evaluation was performed.

3.0 Site Observation

Each significant structure of the water treatment and distribution system was observed on June 13 & 14, 2022. Steve Haney, Utilities Manager, of the City of Sweet Home was present during the site observations. The existing structures were observed for compliance with the original design drawings and details. Deviations from the original design documents were noted. Signs for structural deficiencies or distress were a primary focus and any signs were noted.

4.0 Structure Summaries

4.1 Raw Water Intake

The Raw Water Intake structure is located on Foster Reservoir Dam. The intake structure consists of a slab on grade with CMU block walls supporting a wood framed roof. The structure was built in 2007 and is in good condition. There is no rain gutter on the back side of the mono-sloped roof which as contributed to some minor exposure or scour on the downhill side of the building.

4.2 Raw Water Pump Station

The Raw Water Pump Station is located north of the Water Treatment Plant. The pump station consists of a concrete wet well with a CMU block pump house above approximately 8 feet of the east end. Approximately 16 feet of the pump house consists of a slab on grade with 8 feet being an elevated slab over the wet well. The CMU block walls support a wood framed truss roof. The structure was built in 2008 and is in good condition.

4.3 Water Treatment Building

The Water Treatment Building has a concrete clear well with a concrete slab top below a portion of the building. The remainder of the main floor consists of a slab on grade. The south side of the building is embedded into the hillside and the soil is retained by a concrete retaining wall. The remainder of the perimeter walls were constructed with 10” CMU block. The building is framed by Pre-Engineered Metal Building steel frames with light gauge metal roof purlins. The west portion of the building contains a wood framed mezzanine that contains offices, an IT room, a laboratory, and a meeting room.

The structure was built in 2008 and is in good condition despite some issues. Steven pointed out some insulation that became saturated when condensation building up on the underside of the metal roof. Rust and corrosion was observed near the base of most of the steel columns.

4.4 Water Treatment Pond

The Water Treatment Pond just north of the Water Treatment Building. The Water Treatment Pond is a concrete structure that was built in 2008 and is in good condition.

4.5 Lake Pointe Pump Station

The Lake Pointe Pump Station structure is located on the east side of town just off of Hwy 20 near Foster Reservoir. The pump station consists of a slab on grade with CMU block walls supporting wood framed roof trusses. The structure was built in 2016 and is in good condition.

4.6 Strawberry Pump Station

The Strawberry Pump Station consists of a plastic cover bolted to a concrete pad on grade. The plastic cover protects the pump & electrical panels from weather. The pump station was installed in 2001 and is in good condition.

4.7 Strawberry Reservoir

The Strawberry Reservoir is a bolted steel tank on a concrete foundation on grade that was built in 2001. Steven pointed out that several of the nuts for the anchor bolts are loose. Other than tightening the anchor nuts, the structure is in good condition.

4.8 Strawberry Reservoir Vault

The Strawberry Reservoir has an accessory structure on site. The vault structure consists of a slab on grade with CMU block walls supporting a grating floor and a wood framed roof. The structure was built in 2001 and is in fair condition. Mold, rust and corrosion was observed on the interior of the structure. A fan intended to provide ventilation does not appear to operate properly, if at all.

4.9 10th Avenue Reservoir 300k

The 300k gallon reservoir at 10th Avenue is inactive and is not providing service to the water distribution system. The existing reservoir consists of a concrete slab on grade with concrete walls and a concrete lid. The original drawings from 1938 show a wood framed lid, so at some point the structure was retrofitted. The reservoir is in fair condition.

4.10 10th Avenue Reservoir 700k

The 700k gallon reservoir at 10th Avenue consists of a concrete slab on grade with concrete walls and a concrete lid. The walls have been coated with shotcrete at some point. It is unlikely that the original structure was constructed using shotcrete in 1951. The shotcrete coating may have been used to seal cracks and protect the existing concrete walls, but that is speculation. For a structure originally built in 1951 it is in good condition.

4.11 10th Avenue Reservoir 1.5M

The 1.5M gallon reservoir at 10th Avenue consists of a concrete slab on grade with concrete walls and a concrete lid. Similar to the 700k reservoir, the walls of the 1.5M reservoir have a shotcrete finish. It is possible that the original structure was constructed using shotcrete in 1969. It is also possible that the shotcrete coating may have been used to seal cracks and protect the existing concrete walls, but that is speculation. For a structure originally built in 1969 it is in good condition.

4.12 49th Avenue Reservoir 2.0M

The 2.0M gallon reservoir at 49th Avenue consists of a concrete slab on grade with concrete walls and a concrete lid. Similar to the two previously mentioned reservoirs, the walls of the 2.0M reservoir have a shotcrete finish. It is possible that the original structure was constructed using shotcrete in 1993. It is also possible that the shotcrete coating may have been used to seal cracks and protect the existing concrete walls, but that is speculation. For a structure originally built in 1993 it is in good condition.

5.0 ASCE/SEI 41-17 Tier 1 Checklists, Quick Checks, and Evaluations

The Tier 1 level of the American Society of Civil Engineer's "Seismic Evaluation of Existing Buildings – ASCE 41-17" guideline was used to evaluate each structure. The purpose of a Tier 1 evaluation is to provide "Quick Checks" to evaluate a structure and determine deficiencies related to the lateral resisting elements.

It is the intent of the evaluation to determine the structural deficiencies of each structure as compared to current prescribed loading and detailing requirements for lateral (wind/seismic) loading to a performance level of "Immediate Occupancy" per ASCE 41-17 section 2.3.1.1. The level of performance is defined per ASCE 41-17 as:

"Structural Performance Level S-1, Immediate Occupancy, is defined as the postearthquake damage state in which a structure remains safe to occupy and essentially retains its preearthquake strength and stiffness."

The commentary to ASCE 41-17 section 2.3.1.1 describes the level of performance as:

"Only very limited structural damage has occurred. The basic vertical- and lateral-force-resisting systems of the building retain almost all of the preearthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low, and although some minor structural repairs might be appropriate, these repairs would generally not be required before reoccupancy. Continued use of the building is not limited by its structural condition but might be limited by damage or disruption to nonstructural elements of the building, furnishings, or equipment and availability of external utility services."

ASCE 41-17 requires that a seismic hazard level is determined. In order to obtain a performance level of "Immediate Occupancy" the seismic hazard shall be BSE-1E as defined in section 2.4.1.4 and C2.4.1.4. The BSE-1E hazard level earthquake has a 20% chance of recurring every 50 years. This design level earthquake has a similar rate of occurrence and magnitude as the current state adopted building codes. A 25% reduction in force is recommended by the State of Oregon for seismic rehabilitation grants. The City of Portland City Code for the evaluation and rehabilitation of existing buildings contains similar recommendations. It is likely that this level of earthquake hazard provides an appropriate level of performance for these facilities.

Lateral force resisting systems work in conjunction with gravity framing systems. The existing gravity framing system was also observed for structural distress during the site observation.

ASCE 41-17 requires that non-structural items retain their position during earthquake shaking for structures in order to obtain a performance level of "Immediate Occupancy". Non-structural items include utilities, fixtures, equipment, finishes and furnishings.

The ASCE 41-17 checklists for each structure are included in Appendix A for reference.

6.0 Seismic Rehabilitation Recommendations

The following items summarize the findings and recommendations for structural improvements for each structure. The recommendations are required to resolve structural deficiencies and maintain the load bearing system of each structure. A complete load bearing system that is capable of resisting building code load combinations is important to the continuing performance of each structure.

6.1 Raw Water Intake

The Raw Water Intake structure is considered a Reinforced Masonry Bearing Walls with Flexible Diaphragm (RM1) structure. No deficiencies were found in the checklists for the Raw Water Intake structure. The only non-structural deficiency found during the site observation is:

- Lack of rain gutter on the back side of the roof contributing to some minor exposure or scour on the downhill side of the building.

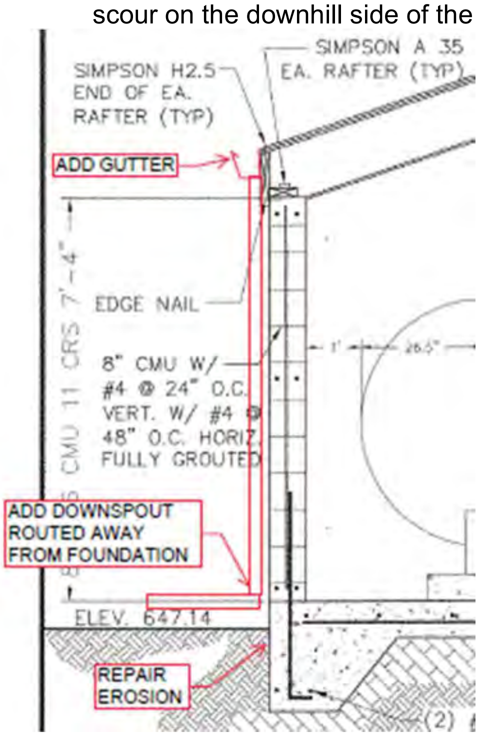


Figure 6.1 Raw Water Intake

6.2 Raw Water Pump Station

The Raw Water Pump Station is considered a Reinforced Masonry Bearing Walls with Flexible Diaphragm (RM1) structure. No deficiencies were found in the checklists, document review and site observation for the Raw Water Pump Station structure.

6.3 Water Treatment Building

The Water Treatment Building is considered a Reinforced Masonry Bearing Walls with Flexible Diaphragm (RM1) structure in the east-west direction and a Metal Building Frame (S3) in the north-south direction. The noncompliant items discovered in the checklists and site observation include:

- REDUNDANCY: The mezzanine is open to the east toward the filters making it a 3 sided diaphragm. No shear walls are provided for lateral resistance of the mezzanine diaphragm along the east side.
- PROPORTIONS: The height to thickness ratio of the masonry walls exceed the recommended limits.
- OPENINGS AT EXTERIOR MASONRY WALLS: The stair opening in the mezzanine diaphragm is adjacent to the exterior masonry wall and exceeds the recommended limits.
- PLAN IRREGULARITIES: The stair opening in the mezzanine diaphragm is considered a plan irregularity. There is a lack of tensile capacity around the stair opening in the mezzanine diaphragm.
- UNBLOCKED DIAPHRAGMS: The mezzanine diaphragm was not noted to have blocking at the plywood panel edges. The unblocked diaphragm exceeds allowable limits and aspect ratios when subject to east-west lateral loading.
- SUSPENDED CONTENTS: Several items are suspended from the structure and are free to swing or move but may damage themselves or adjoining components.
- TALL NARROW EQUIPMENT: There are several pieces of equipment more than 6 feet tall that should be anchored to the floor or adjacent walls.
- CONDUIT COUPLINGS: Conduit greater than 2.5 inches should have flexible couplings.
- The condensation buildup above the insulation should be addressed to prevent further failure of the insulation.
- The rust and corrosion around the base of the steel columns should be treated, repaired and properly coated to prevent further deterioration.

Fig 6.3.1 Open Mezzanine Lacks Redundancy

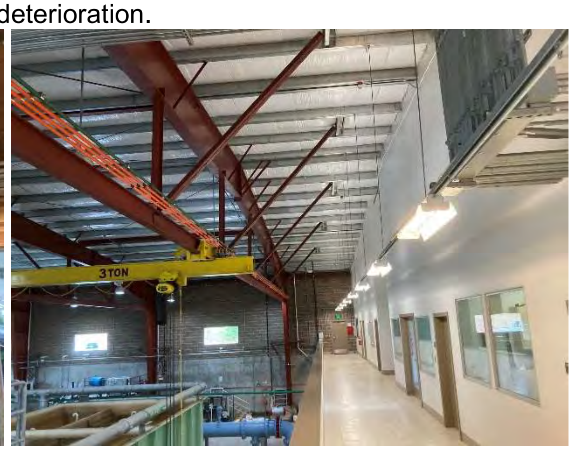


Figure 6.3.2 Lights & Conduits at Egress

Figure 6.3.3 Corrosion at Steel Columns

Figure 6.3.4 Tanks without Restraints

6.4 Water Treatment Pond

The Water Treatment Pond is considered a Concrete Shear Wall (C2) structure. No deficiencies were found in the checklists, document review and site observation for the Water Treatment Pond structure.

6.5 Lake Pointe Pump Station

The Lake Pointe Pump Station is considered a Reinforced Masonry Bearing Walls with Flexible Diaphragm (RM1) structure. No deficiencies were found in the checklists, document review and site observation for the Lake Pointe Pump Station structure.

6.6 Strawberry Pump Station

The Strawberry Pump Station is an unclassified structure. No deficiencies were found in the checklists, document review and site observation.

6.7 Strawberry Reservoir

The Strawberry Reservoir is considered a Steel Plate Shear Wall (S6) structure. No deficiencies were found in the checklists, document review. The only item to be addressed from the site observation is:

- Tighten the nuts of the existing anchor bolts.

Figure 6.7 Strawberry Reservoir Anchor Bolts

6.8 Strawberry Reservoir Vault

The Strawberry Reservoir is considered a Reinforced Masonry Bearing Walls with Flexible Diaphragm (RM1) structure. No deficiencies were found in the checklists, document review. The items to be addressed from the site observation include:

- Repair the fan or provide adequate ventilation to prevent future build up of mold, rust and corrosion
- Clean and repair the mold, rust and corrosion to original condition.


Figure 6.8.1 Strawberry Vault


Figure 6.8.2 Strawberry Vault Corrosion

6.9 10th Avenue Reservoir 300k

The 300k gallon reservoir at 10th Avenue is considered a Concrete Shear Wall (C2) structure. No deficiencies were found in the checklists, document review and site observation.

6.10 10th Avenue Reservoir 700k

The 700k gallon reservoir at 10th Avenue is considered a Concrete Shear Wall (C2) structure. No deficiencies were found in the checklists, document review and site observation.

6.11 10th Avenue Reservoir 1.5M

The 1.5M gallon reservoir at 10th Avenue is considered a Concrete Shear Wall (C2) structure. The noncompliant items discovered in the checklists and site observation include:

- REINFORCING STEEL: The amount of vertical reinforcing steel bars in the existing concrete walls is less than the recommended amount.
- WALL THICKNESS: The perimeter wall thickness exceeds the recommended limit for the unsupported height of the reservoir.

6.12 49th Avenue Reservoir 2.0M

The 2.0M gallon reservoir at 49th Avenue is considered a Concrete Shear Wall (C2) structure. The only noncompliant item discovered in the checklists and site observation include:

- WALL THICKNESS: The perimeter wall thickness exceeds the recommended limit for the unsupported height of the reservoir.

Figure 6.12 49th Avenue Reservoir 2.0M Wall

6.13 General nonstructural items.

It is recommended that City staff review the Nonstructural Checklist and consider the items at each facility for compliance with the best practices for storing items and equipment. Some conditions to consider include:

- FIRE SUPPRESSION PIPING: Make sure piping is anchored and braced in accordance with current NFPA standards. Consider anchoring and bracing all piping in all facilities.
- HAZARDOUS MATERIAL STORAGE: Some chemicals used in the treatment process or used during regular cleaning and maintenance processes may be considered hazardous when spilled. Items storing these chemicals should be restrained to prevent displacement, tipping or falling.
- HAZARDOUS MATERIAL DISTRIBUTION: Natural gas piping should anchored or braced adequately to prevent damage that might allow the hazardous material to release.
- SHUTOFF VALVES: Piping containing hazardous material, including natural gas, should have shutoff valves or other devices to prevent spills or leaks.
- FLEXIBLE COUPLINGS: Hazardous material, ductwork and piping, including natural gas piping, should have flexible couplings.
- LIGHT FIXTURES LENSE COVERS: Make sure lens covers on light fixtures are attached with safety devices and add safety devices if necessary.
- INDUSTRIAL STORAGE RACKS: Industrial storage racks or similar items that are more than 12 feet high should be anchored to the floor.
- TALL NARROW CABINETS: Cabinets, lockers, bookshelves, etc. more than 6 feet high and with height-to-depth ratios exceeding 3:1 should anchored to the floor or wall.
- FALL-PRONE CONTENTS: Equipment, stored items weighing more than 20 pounds and more than 4 feet above the floor should be braced or restrained.
- FALL-PRONE EQUIPMENT: Equipment weighing more than 20 pounds and more than 4 feet above the floor should be braced or restrained.
- IN-LINE EQUIPMENT: Equipment installed in line with a duct or piping system, with an operating weight more than 75 pounds should be laterally braced independent of the duct or piping system.
- TALL NARROW EQUIPMENT: Equipment, tanks, etc. more than 6 feet high and with height-to-depth ratios exceeding 3:1 should anchored to the floor or wall.
- SUSPENDED EQUIPMENT: Equipment suspended without lateral bracing should be free to swing or move with the structure without damaging itself or adjoining components.
- HEAVY EQUIPMENT: Floor supported or platform supported equipment weighing more than 400 pounds should be anchored to the structure.
- CONDUIT COUPLINGS: Conduit greater than 2.5 inches should have flexible couplings.
- FLEXIBLE COUPLINGS: Fluid and gas piping should have flexible couplings.
- FLUID AND GAS PIPING: Fluid and gas piping should be anchored and braced to the structure to limit spills or leaks.

Based on previous experience and observations at site the buildings may contain some form of hazardous material. These materials will need to be dealt with on a case-by-case basis as they are encountered during the project.

7.0 Conclusions

The majority of the Sweet Home water treatment and distribution system is in reasonable structural condition. Maintenance and structural upgrades should be part of the City's operating plan. Replacement of aging structures should also be included in the City's long term plan regardless of physical condition.

8.0 Limitations

This Structural Technical Memorandum has been prepared for the City of Sweet Home Water Master Plan. The conclusions and recommendations in this memorandum were derived from the professional review of documentation that was provided by the City of Sweet Home, West Yost, published literature and limited site observations. ACE Engineering is not responsible for errors and omissions that might exist in documents and construction performed by others.

This report has been completed within the limitation of the West Yost approved scope of work. The services provided have been performed in a manner consistent with the level of competency presently maintained by other practicing professional engineers in the same type of work in the community of the project for the professional and technical soundness, accuracy, and adequacy of the work. ACE Engineering is not responsible for the use of this report for anything other than the Sweet Home Water Master Plan.

ACE ENGINEERING LLC

REGISTERED PROFESSIONAL ENGINEER
64230PE
OREGON
MAY 9, 2000
ALLAN T GOFFE
EXPIRES 6/30/2025

Allan T Goffe, P.E., S.E.
Principle Engineer

ACE ENGINEERING LLC
professional structural engineering
commercial . residential . industrial

po box 231 . ashland . oregon 97520
541.552.1417 . ace-engineeringllc.com

APPENDIX A - **ASCE/SEI 41-17 CHECKLISTS**

Table 17-3. Immediate Occupancy Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Building System—General			
C NC N/A U	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation.	5.4.1.1	A.2.1.1
C NC N/A U	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.5% of the height of the shorter building in low seismicity, 1.0% in moderate seismicity, and 3.0% in high seismicity.	5.4.1.2	A.2.1.2
C NC N/A U	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure.	5.4.1.3	A.2.1.3
Building System—Building Configuration			
C NC N/A U	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above.	5.4.2.1	A.2.2.2
C NC N/A U	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above.	5.4.2.2	A.2.2.3
C NC N/A U	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation.	5.4.2.3	A.2.2.4
C NC N/A U	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines.	5.4.2.4	A.2.2.5
C NC N/A U	MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered.	5.4.2.5	A.2.2.6

continues

Table 17-3 *(Continued)*. Immediate Occupancy Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension.	5.4.2.6	A.2.2.7
Low Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Geologic Site Hazards			
C NC N/A U	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building.	5.4.3.1	A.6.1.1
C NC N/A U	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure.	5.4.3.1	A.6.1.2
C NC N/A U	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated.	5.4.3.1	A.6.1.3
Moderate and High Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Foundation Configuration			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1
C NC N/A U	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C.	5.4.3.4	A.6.2.2

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

FOSTER DAM RAW WATER INTAKE STRUCTURE

Table 17-35. Immediate Occupancy Structural Checklist for Building Types RM1 and RM2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 70 lb/in.^{2} (4.83 MPa).	5.5.3.1.1	A.3.2.4.1
C NC N/A U	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in., and all vertical bars extend to the top of the walls.	5.5.3.1.3	A.3.2.4.2
Connections			
C NC N/A U	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers.	5.7.1.3	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1

continues

Table 17-35 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types RM1 and RM2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation.	5.7.3.4	A.5.3.5
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
Stiff Diaphragms			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab.	5.6.4	A.4.5.1
C NC N/A U	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements.	5.7.2	A.5.2.3
Foundation System			
C NC N/A U	DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil.		A.6.2.3
C NC N/A U	SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story.		A.6.2.4
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	REINFORCING AT WALL OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides.	5.5.3.1.5	A.3.2.4.3
C NC N/A U	PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than 30.	5.5.3.1.2	A.3.2.4.4
Diaphragms (Stiff or Flexible)			
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 4 ft (1.2 m) long.	5.6.1.3	A.4.1.6
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.3
C NC N/A U	NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors.	5.7.1.2	A.5.1.4

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

WATER TREATMENT PLANT RAW WATER INTAKE

Table 17-35. Immediate Occupancy Structural Checklist for Building Types RM1 and RM2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 70 lb/in.^{2} (4.83 MPa).	5.5.3.1.1	A.3.2.4.1
C NC N/A U	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in., and all vertical bars extend to the top of the walls.	5.5.3.1.3	A.3.2.4.2
Connections			
C NC N/A U	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers.	5.7.1.3	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1

continues

Table 17-35 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types RM1 and RM2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation.	5.7.3.4	A.5.3.5
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
Stiff Diaphragms			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab.	5.6.4	A.4.5.1
C NC N/A U	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements.	5.7.2	A.5.2.3
Foundation System			
C NC N/A U	DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil.		A.6.2.3
C NC N/A U	SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story.		A.6.2.4
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	REINFORCING AT WALL OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides.	5.5.3.1.5	A.3.2.4.3
C NC N/A U	PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than 30.	5.5.3.1.2	A.3.2.4.4
Diaphragms (Stiff or Flexible)			
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 4 ft (1.2 m) long.	5.6.1.3	A.4.1.6
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.3
C NC N/A U	NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors.	5.7.1.2	A.5.1.4

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

WATER TREATMENT PLANT

Table 17-35. Immediate Occupancy Structural Checklist for Building Types RM1 and RM2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. MEZZANINE	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 70 lb/in.^{2} (4.83 MPa).	5.5.3.1.1	A.3.2.4.1
C NC N/A U	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in., and all vertical bars extend to the top of the walls.	5.5.3.1.3	A.3.2.4.2
Connections			
C NC N/A U	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers.	5.7.1.3	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1

continues

Table 17-35 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types RM1 and RM2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation.	5.7.3.4	A.5.3.5
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
Stiff Diaphragms			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab.	5.6.4	A.4.5.1
C NC N/A U	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements.	5.7.2	A.5.2.3
Foundation System			
C NC N/A U	DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil.		A.6.2.3
C NC N/A U	SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story.		A.6.2.4
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	REINFORCING AT WALL OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides.	5.5.3.1.5	A.3.2.4.3
C NC N/A U	PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than 30. 35.8	5.5.3.1.2	A.3.2.4.4
Diaphragms (Stiff or Flexible)			
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 4 ft (1.2 m) long.	5.6.1.3 MEZZANINE	A.4.1.6
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1.	5.6.2 MEZZANINE	A.4.2.3
C NC N/A U	NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors.	5.7.1.2	A.5.1.4

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

WATER TREATMENT PLANT

Table 17-13. Immediate Occupancy Checklist for Building Type S3

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low and Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	BRACE AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4.4.3.4, is less than $0.50F_y$.	5.5.4.1	A.3.3.1.2
C NC N/A U	FLEXURAL STRESS CHECK: The average flexural stress in the moment-frame columns and beams, calculated using the Quick Check procedure of Section 4.4.3.9, is less than F_y.	5.5.2.1.2	A.3.1.3.3
Connections			
C NC N/A U	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel moment frames.	5.7.2	A.5.2.2
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation.	5.7.3.1	A.5.3.1
Moderate Seismicity (Complete the Following Items in Addition to the Items for Very Low and Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the elastic moment (F_yS) of the adjoining members.	5.5.2.2.1	A.3.1.3.4
Diaphragms			
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7

continues

Table 17-13 *(Continued)*. Immediate Occupancy Checklist for Building Type S3

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	ROOF PANELS: Where considered as diaphragm elements for lateral resistance, metal, plastic, or cementitious roof panels are positively attached to the roof framing to resist seismic forces.	5.7.5	A.5.5.1
C NC N/A U	WALL PANELS: Where considered as shear elements for lateral resistance, metal, fiberglass, or cementitious wall panels are positively attached to the framing and foundation to resist seismic forces.	5.7.5	A.5.5.2
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones.	5.5.2.2.1	A.3.1.3.4
C NC N/A U	COMPACT MEMBERS: All frame elements meet compact section requirements in accordance with AISC 360, Table B4.1.	5.5.2.2.4	A.3.1.3.8
C NC N/A U	BEAM PENETRATIONS: All openings in frame-beam webs are less than one quarter of the beam depth and are located in the center half of the beams.	5.5.2.2.5	A.3.1.3.9
C NC N/A U	OUT-OF-PLANE BRACING: Beam–column joints are braced out of plane.	5.5.2.2.7	A.3.1.3.11
C NC N/A U	BOTTOM FLANGE BRACING: The bottom flanges of beams are braced out of plane.	5.5.2.2.8	A.3.1.3.12
Connections			
C NC N/A U	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel moment frames, and the connections are able to develop the lesser of the strength of the frames or the diaphragms.	5.7.2	A.5.2.2
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation, and the anchorage is able to develop the least of the following: the tensile capacity of the column, the tensile capacity of the lowest level column splice (if any), or the uplift capacity of the foundation.	5.7.3.1	A.5.3.1
Foundation System			
C NC N/A U	DEEP FOUNDATIONS: Piles and piers are capable of transferring the seismic forces between the structure and the soil.		A.6.2.3
C NC N/A U	SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story.		A.6.2.4

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

LAKE POINT PUMP STATION

Table 17-35. Immediate Occupancy Structural Checklist for Building Types RM1 and RM2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 70 lb/in.^{2} (4.83 MPa).	5.5.3.1.1	A.3.2.4.1
C NC N/A U	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in., and all vertical bars extend to the top of the walls.	5.5.3.1.3	A.3.2.4.2
Connections			
C NC N/A U	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers.	5.7.1.3	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1

continues

Table 17-35 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types RM1 and RM2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation.	5.7.3.4	A.5.3.5
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
Stiff Diaphragms			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab.	5.6.4	A.4.5.1
C NC N/A U	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements.	5.7.2	A.5.2.3
Foundation System			
C NC N/A U	DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil.		A.6.2.3
C NC N/A U	SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story.		A.6.2.4
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	REINFORCING AT WALL OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides.	5.5.3.1.5	A.3.2.4.3
C NC N/A U	PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than 30.	5.5.3.1.2	A.3.2.4.4
Diaphragms (Stiff or Flexible)			
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 4 ft (1.2 m) long.	5.6.1.3	A.4.1.6
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.3
C NC N/A U	NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors.	5.7.1.2	A.5.1.4

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

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Table 17-24. Collapse Prevention Structural Checklist for Building Types C2 and C2a

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system.	5.5.2.5.1	A.3.1.6.1
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.2.1
C NC N/A U	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction.	5.5.3.1.3	A.3.2.2.2
Connections			
C NC N/A U	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing directly above the foundation.	5.7.3.4	A.5.3.5
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3
C NC N/A U	COUPLING BEAMS: The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning.	5.5.3.2.1	A.3.2.2.3
Diaphragms (Stiff or Flexible)			
C NC N/A U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps.	5.7.3.5	A.5.3.8

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-25. Immediate Occupancy Structural Checklist for Building Types C2 and C2a

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system.	5.5.2.5.1	A.3.1.6.1
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.2.1
C NC N/A U	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. The spacing of reinforcing steel is equal to or less than 18 in. (457 mm).	5.5.3.1.3	A.3.2.2.2
Connections			
C NC N/A U	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of loads to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation.	5.7.3.4	A.5.3.5
Foundation System			
C NC N/A U	DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil.		A.6.2.3
C NC N/A U	SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story.		A.6.2.4
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components and are compliant with the following items in Table 17-23: COLUMN-BAR SPLICES, BEAM-BAR SPLICES, COLUMN-TIE SPACING, STIRRUP SPACING, and STIRRUP AND TIE HOOKS.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	FLAT SLABS: Flat slabs or plates not part of seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3
C NC N/A U	COUPLING BEAMS: The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. Coupling beams have the capacity in shear to develop the uplift capacity of the adjacent wall.	5.5.3.2.1	A.3.2.2.3
C NC N/A U	OVERTURNING: All shear walls have aspect ratios less than 4-to-1. Wall piers need not be considered.	5.5.3.1.4	A.3.2.2.4
C NC N/A U	CONFINEMENT REINFORCING: For shear walls with aspect ratios greater than 2-to-1, the boundary elements are confined with spirals or ties with spacing less than $8d_b$.	5.5.3.2.2	A.3.2.2.5
C NC N/A U	WALL REINFORCING AT OPENINGS: There is added trim reinforcement around all wall openings with a dimension greater than three times the thickness of the wall.	5.5.3.1.5	A.3.2.2.6
C NC N/A U	WALL THICKNESS: Thicknesses of bearing walls are not less than 1/25 the unsupported height or length, whichever is shorter, nor less than 4 in. (101 mm).	5.5.3.1.2	A.3.2.2.7

continues

Table 17-25 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types C2 and C2a

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Diaphragms (Stiff or Flexible)			
C NC N/A U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.3
C NC N/A U	NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps; the pile cap reinforcement and pile anchorage are able to develop the tensile capacity of the piles.	5.7.3.5	A.5.3.8

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

10TH STREET RESERVOIR - 1938 TANK

Table 17-25. Immediate Occupancy Structural Checklist for Building Types C2 and C2a

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system.	5.5.2.5.1	A.3.1.6.1
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.2.1
C NC N/A U	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. The spacing of reinforcing steel is equal to or less than 18 in. (457 mm).	5.5.3.1.3	A.3.2.2.2
Connections			
C NC N/A U	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of loads to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation.	5.7.3.4	A.5.3.5
Foundation System			
C NC N/A U	DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil.		A.6.2.3
C NC N/A U	SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story.		A.6.2.4
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components and are compliant with the following items in Table 17-23: COLUMN-BAR SPLICES, BEAM-BAR SPLICES, COLUMN-TIE SPACING, STIRRUP SPACING, and STIRRUP AND TIE HOOKS.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	FLAT SLABS: Flat slabs or plates not part of seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3
C NC N/A U	COUPLING BEAMS: The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. Coupling beams have the capacity in shear to develop the uplift capacity of the adjacent wall.	5.5.3.2.1	A.3.2.2.3
C NC N/A U	OVERTURNING: All shear walls have aspect ratios less than 4-to-1. Wall piers need not be considered.	5.5.3.1.4	A.3.2.2.4
C NC N/A U	CONFINEMENT REINFORCING: For shear walls with aspect ratios greater than 2-to-1, the boundary elements are confined with spirals or ties with spacing less than $8d_b$.	5.5.3.2.2	A.3.2.2.5
C NC N/A U	WALL REINFORCING AT OPENINGS: There is added trim reinforcement around all wall openings with a dimension greater than three times the thickness of the wall.	5.5.3.1.5	A.3.2.2.6
C NC N/A U	WALL THICKNESS: Thicknesses of bearing walls are not less than 1/25 the unsupported height or length, whichever is shorter, nor less than 4 in. (101 mm).	5.5.3.1.2	A.3.2.2.7

continues

Table 17-25 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types C2 and C2a

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Diaphragms (Stiff or Flexible)			
C NC N/A U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.3
C NC N/A U	NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps; the pile cap reinforcement and pile anchorage are able to develop the tensile capacity of the piles.	5.7.3.5	A.5.3.8

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

10TH STREET RESERVOIR - 1951 TANK

Table 17-25. Immediate Occupancy Structural Checklist for Building Types C2 and C2a

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system.	5.5.2.5.1	A.3.1.6.1
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.2.1
C NC N/A U	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. The spacing of reinforcing steel is equal to or less than 18 in. (457 mm).	5.5.3.1.3	A.3.2.2.2
Connections			
C NC N/A U	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of loads to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation.	5.7.3.4	A.5.3.5
Foundation System			
C NC N/A U	DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil.		A.6.2.3
C NC N/A U	SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story.		A.6.2.4
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components and are compliant with the following items in Table 17-23: COLUMN-BAR SPLICES, BEAM-BAR SPLICES, COLUMN-TIE SPACING, STIRRUP SPACING, and STIRRUP AND TIE HOOKS.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	FLAT SLABS: Flat slabs or plates not part of seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3
C NC N/A U	COUPLING BEAMS: The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. Coupling beams have the capacity in shear to develop the uplift capacity of the adjacent wall.	5.5.3.2.1	A.3.2.2.3
C NC N/A U	OVERTURNING: All shear walls have aspect ratios less than 4-to-1. Wall piers need not be considered.	5.5.3.1.4	A.3.2.2.4
C NC N/A U	CONFINEMENT REINFORCING: For shear walls with aspect ratios greater than 2-to-1, the boundary elements are confined with spirals or ties with spacing less than $8d_b$.	5.5.3.2.2	A.3.2.2.5
C NC N/A U	WALL REINFORCING AT OPENINGS: There is added trim reinforcement around all wall openings with a dimension greater than three times the thickness of the wall.	5.5.3.1.5	A.3.2.2.6
C NC N/A U	WALL THICKNESS: Thicknesses of bearing walls are not less than 1/25 the unsupported height or length, whichever is shorter, nor less than 4 in. (101 mm).	5.5.3.1.2	A.3.2.2.7

continues

Table 17-25 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types C2 and C2a

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Diaphragms (Stiff or Flexible)			
C NC N/A U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.3
C NC N/A U	NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps; the pile cap reinforcement and pile anchorage are able to develop the tensile capacity of the piles.	5.7.3.5	A.5.3.8

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

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Table 17-25. Immediate Occupancy Structural Checklist for Building Types C2 and C2a

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system.	5.5.2.5.1	A.3.1.6.1
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.2.1
C NC N/A U	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. The spacing of reinforcing steel is equal to or less than 18 in. (457 mm).	5.5.3.1.3	A.3.2.2.2
Connections			
C NC N/A U	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of loads to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation.	5.7.3.4	A.5.3.5
Foundation System			
C NC N/A U	DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil.		A.6.2.3
C NC N/A U	SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story.		A.6.2.4
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components and are compliant with the following items in Table 17-23: COLUMN-BAR SPLICES, BEAM-BAR SPLICES, COLUMN-TIE SPACING, STIRRUP SPACING, and STIRRUP AND TIE HOOKS.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	FLAT SLABS: Flat slabs or plates not part of seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3
C NC N/A U	COUPLING BEAMS: The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. Coupling beams have the capacity in shear to develop the uplift capacity of the adjacent wall.	5.5.3.2.1	A.3.2.2.3
C NC N/A U	OVERTURNING: All shear walls have aspect ratios less than 4-to-1. Wall piers need not be considered.	5.5.3.1.4	A.3.2.2.4
C NC N/A U	CONFINEMENT REINFORCING: For shear walls with aspect ratios greater than 2-to-1, the boundary elements are confined with spirals or ties with spacing less than $8d_b$.	5.5.3.2.2	A.3.2.2.5
C NC N/A U	WALL REINFORCING AT OPENINGS: There is added trim reinforcement around all wall openings with a dimension greater than three times the thickness of the wall.	5.5.3.1.5	A.3.2.2.6
C NC N/A U	WALL THICKNESS: Thicknesses of bearing walls are not less than 1/25 the unsupported height or length, whichever is shorter, nor less than 4 in. (101 mm).	5.5.3.1.2	A.3.2.2.7

continues

Table 17-25 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types C2 and C2a

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Diaphragms (Stiff or Flexible)			
C NC N/A U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.3
C NC N/A U	NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps; the pile cap reinforcement and pile anchorage are able to develop the tensile capacity of the piles.	5.7.3.5	A.5.3.8

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

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Table 17-25. Immediate Occupancy Structural Checklist for Building Types C2 and C2a

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Very Low Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system.	5.5.2.5.1	A.3.1.6.1
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.2.1
C NC N/A U	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. The spacing of reinforcing steel is equal to or less than 18 in. (457 mm).	5.5.3.1.3	A.3.2.2.2
Connections			
C NC N/A U	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of loads to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation.	5.7.3.4	A.5.3.5
Foundation System			
C NC N/A U	DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil.		A.6.2.3
C NC N/A U	SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story.		A.6.2.4
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components and are compliant with the following items in Table 17-23: COLUMN-BAR SPLICES, BEAM-BAR SPLICES, COLUMN-TIE SPACING, STIRRUP SPACING, and STIRRUP AND TIE HOOKS.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	FLAT SLABS: Flat slabs or plates not part of seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3
C NC N/A U	COUPLING BEAMS: The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. Coupling beams have the capacity in shear to develop the uplift capacity of the adjacent wall.	5.5.3.2.1	A.3.2.2.3
C NC N/A U	OVERTURNING: All shear walls have aspect ratios less than 4-to-1. Wall piers need not be considered.	5.5.3.1.4	A.3.2.2.4
C NC N/A U	CONFINEMENT REINFORCING: For shear walls with aspect ratios greater than 2-to-1, the boundary elements are confined with spirals or ties with spacing less than $8d_b$.	5.5.3.2.2	A.3.2.2.5
C NC N/A U	WALL REINFORCING AT OPENINGS: There is added trim reinforcement around all wall openings with a dimension greater than three times the thickness of the wall.	5.5.3.1.5	A.3.2.2.6
C NC N/A U	WALL THICKNESS: Thicknesses of bearing walls are not less than 1/25 the unsupported height or length, whichever is shorter, nor less than 4 in. (101 mm).	5.5.3.1.2	A.3.2.2.7

continues

Table 17-25 *(Continued)*. Immediate Occupancy Structural Checklist for Building Types C2 and C2a

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Diaphragms (Stiff or Flexible)			
C NC N/A U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.3
C NC N/A U	NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps; the pile cap reinforcement and pile anchorage are able to develop the tensile capacity of the piles.	5.7.3.5	A.5.3.8

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-38. Nonstructural Checklist

Status	Evaluation Statement[a,b]	Tier 2 Reference	Commentary Reference
Life Safety Systems			
C NC N/A U	**HR—not required; LS—LMH; PR—LMH**. FIRE SUPPRESSION PIPING: Fire suppression piping is anchored and braced in accordance with NFPA-13.	13.7.4	A.7.13.1
C NC N/A U	**HR—not required; LS—LMH; PR—LMH**. FLEXIBLE COUPLINGS: Fire suppression piping has flexible couplings in accordance with NFPA-13.	13.7.4	A.7.13.2
C NC N/A U	**HR—not required; LS—LMH; PR—LMH**. EMERGENCY POWER: Equipment used to power or control Life Safety systems is anchored or braced.	13.7.7	A.7.12.1
C NC N/A U	**HR—not required; LS—LMH; PR—LMH**. STAIR AND SMOKE DUCTS: Stair pressurization and smoke control ducts are braced and have flexible connections at seismic joints.	13.7.6	A.7.14.1
C NC N/A U	**HR—not required; LS—MH; PR—MH**. SPRINKLER CEILING CLEARANCE: Penetrations through panelized ceilings for fire suppression devices provide clearances in accordance with NFPA-13.	13.7.4	A.7.13.3
C NC N/A U	**HR—not required; LS—not required; PR—LMH**. EMERGENCY LIGHTING: Emergency and egress lighting equipment is anchored or braced.	13.7.9	A.7.3.1
Hazardous Materials			
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH**. HAZARDOUS MATERIAL EQUIPMENT: Equipment mounted on vibration isolators and containing hazardous material is equipped with restraints or snubbers.	13.7.1	A.7.12.2
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH**. HAZARDOUS MATERIAL STORAGE: Breakable containers that hold hazardous material, including gas cylinders, are restrained by latched doors, shelf lips, wires, or other methods.	13.8.3	A.7.15.1
C NC N/A U	**HR—MH; LS—MH; PR—MH**. HAZARDOUS MATERIAL DISTRIBUTION: Piping or ductwork conveying hazardous materials is braced or otherwise protected from damage that would allow hazardous material release.	13.7.3 13.7.5	A.7.13.4
C NC N/A U	**HR—MH; LS—MH; PR—MH**. SHUTOFF VALVES: Piping containing hazardous material, including natural gas, has shutoff valves or other devices to limit spills or leaks.	13.7.3 13.7.5	A.7.13.3
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH**. FLEXIBLE COUPLINGS: Hazardous material ductwork and piping, including natural gas piping, have flexible couplings.	13.7.3 13.7.5	A.7.15.4
C NC N/A U	**HR—MH; LS—MH; PR—MH**. PIPING OR DUCTS CROSSING SEISMIC JOINTS: Piping or ductwork carrying hazardous material that either crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements.	13.7.3 13.7.5 13.7.6	A.7.13.6
Partitions			
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH**. UNREINFORCED MASONRY: Unreinforced masonry or hollow-clay tile partitions are braced at a spacing of at most 10 ft (3.0 m) in Low or Moderate Seismicity, or at most 6 ft (1.8 m) in High Seismicity.	13.6.2	A.7.1.1
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH**. HEAVY PARTITIONS SUPPORTED BY CEILINGS: The tops of masonry or hollow-clay tile partitions are not laterally supported by an integrated ceiling system.	13.6.2	A.7.2.1
C NC N/A U	**HR—not required; LS—MH; PR—MH**. DRIFT: Rigid cementitious partitions are detailed to accommodate the following drift ratios: in steel moment frame, concrete moment frame, and wood frame buildings, 0.02; in other buildings, 0.005.	13.6.2	A.7.1.2
C NC N/A U	**HR—not required; LS—not required; PR—MH**. LIGHT PARTITIONS SUPPORTED BY CEILINGS: The tops of gypsum board partitions are not laterally supported by an integrated ceiling system.	13.6.2	A.7.2.1
C NC N/A U	**HR—not required; LS—not required; PR—MH**. STRUCTURAL SEPARATIONS: Partitions that cross structural separations have seismic or control joints.	13.6.2	A.7.1.3

continues

Table 17-38 *(Continued)*. Nonstructural Checklist

Status	Evaluation Statement[a,b]	Tier 2 Reference	Commentary Reference
C NC N/A U	**HR—not required; LS—not required; PR—MH**. TOPS: The tops of ceiling-high framed or panelized partitions have lateral bracing to the structure at a spacing equal to or less than 6 ft (1.8 m).	13.6.2	A.7.1.4
Ceilings			
C NC N/A U	**HR—H; LS—MH; PR—LMH**. SUSPENDED LATH AND PLASTER: Suspended lath and plaster ceilings have attachments that resist seismic forces for every 12 ft^2 (1.1 m^2) of area.	13.6.4	A.7.2.3
C NC N/A U	**HR—not required; LS—MH; PR—LMH**. SUSPENDED GYPSUM BOARD: Suspended gypsum board ceilings have attachments that resist seismic forces for every 12 ft^2 (1.1 m^2) of area.	13.6.4	A.7.2.3
C NC N/A U	**HR—not required; LS—not required; PR—MH**. INTEGRATED CEILINGS: Integrated suspended ceilings with continuous areas greater than 144 ft^2 (13.4 m^2) and ceilings of smaller areas that are not surrounded by restraining partitions are laterally restrained at a spacing no greater than 12 ft (3.6 m) with members attached to the structure above. Each restraint location has a minimum of four diagonal wires and compression struts, or diagonal members capable of resisting compression.	13.6.4	A.7.2.2
C NC N/A U	**HR—not required; LS—not required; PR—MH**. EDGE CLEARANCE: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 (13.4 m^2) have clearances from the enclosing wall or partition of at least the following: in Moderate Seismicity, 1/2 in. (13 mm); in High Seismicity, 3/4 in. (19 mm).	13.6.4	A.7.2.4
C NC N/A U	**HR—not required; LS—not required; PR—MH**. CONTINUITY ACROSS STRUCTURE JOINTS: The ceiling system does not cross any seismic joint and is not attached to multiple independent structures.	13.6.4	A.7.2.5
C NC N/A U	**HR—not required; LS—not required; PR—H**. EDGE SUPPORT: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 (13.4 m^2) are supported by closure angles or channels not less than 2 in. (51 mm) wide.	13.6.4	A.7.2.6
C NC N/A U	**HR—not required; LS—not required; PR—H**. SEISMIC JOINTS: Acoustical tile or lay-in panel ceilings have seismic separation joints such that each continuous portion of the ceiling is no more than 2,500 ft^2 (232.3 m^2) and has a ratio of long-to-short dimension no more than 4-to-1.	13.6.4	A.7.2.7
Light Fixtures			
C NC N/A U	**HR—not required; LS—MH; PR—MH**. INDEPENDENT SUPPORT: Light fixtures that weigh more per square foot than the ceiling they penetrate are supported independent of the grid ceiling suspension system by a minimum of two wires at diagonally opposite corners of each fixture.	13.6.4 13.7.9	A.7.3.2
C NC N/A U	**HR—not required; LS—not required; PR—H**. PENDANT SUPPORTS: Light fixtures on pendant supports are attached at a spacing equal to or less than 6 ft. Unbraced suspended fixtures are free to allow a 360-degree range of motion at an angle not less than 45 degrees from horizontal without contacting adjacent components. Alternatively, if rigidly supported and/or braced, they are free to move with the structure to which they are attached without damaging adjoining components. Additionally, the connection to the structure is capable of accommodating the movement without failure.	13.7.9	A.7.3.3
C NC N/A U	**HR—not required; LS—not required; PR—H**. LENS COVERS: Lens covers on light fixtures are attached with safety devices.	13.7.9	A.7.3.4
Cladding and Glazing			
C NC N/A U	**HR—MH; LS—MH; PR—MH**. CLADDING ANCHORS: Cladding components weighing more than 10 lb/ft^2 (0.48 kN/m^2) are mechanically anchored to the structure at a spacing equal to or less than the following: for Life Safety in Moderate Seismicity, 6 ft (1.8 m); for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 ft (1.2 m)	13.6.1	A.7.4.1

continues

Table 17-38 *(Continued)*. Nonstructural Checklist

Status	Evaluation Statement[a,b]	Tier 2 Reference	Commentary Reference
C NC N/A U	**HR—not required; LS—MH; PR—MH**. CLADDING ISOLATION: For steel or concrete moment-frame buildings, panel connections are detailed to accommodate a story drift ratio by the use of rods attached to framing with oversize holes or slotted holes of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02, and the rods have a length-to-diameter ratio of 4.0 or less.	13.6.1	A.7.4.3
C NC N/A U	**HR—MH; LS—MH; PR—MH**. MULTI-STORY PANELS: For multi-story panels attached at more than one floor level, panel connections are detailed to accommodate a story drift ratio by the use of rods attached to framing with oversize holes or slotted holes of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02, and the rods have a length-to-diameter ratio of 4.0 or less.	13.6.1	A.7.4.4
C NC N/A U	**HR—not required; LS—MH; PR—MH**. THREADED RODS: Threaded rods for panel connections detailed to accommodate drift by bending of the rod have a length-to-diameter ratio greater than 0.06 times the story height in inches for Life Safety in Moderate Seismicity and 0.12 times the story height in inches for Life Safety in High Seismicity and Position Retention in any seismicity.	13.6.1	A.7.4.9
C NC N/A U	**HR—MH; LS—MH; PR—MH**. PANEL CONNECTIONS: Cladding panels are anchored out of plane with a minimum number of connections for each wall panel, as follows: for Life Safety in Moderate Seismicity, 2 connections; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 connections.	13.6.1.4	A.7.4.5
C NC N/A U	**HR—MH; LS—MH; PR—MH**. BEARING CONNECTIONS: Where bearing connections are used, there is a minimum of two bearing connections for each cladding panel.	13.6.1.4	A.7.4.6
C NC N/A U	**HR—MH; LS—MH; PR—MH**. INSERTS: Where concrete cladding components use inserts, the inserts have positive anchorage or are anchored to reinforcing steel.	13.6.1.4	A.7.4.7
C NC N/A U	**HR—not required; LS—MH; PR—MH**. OVERHEAD GLAZING: Glazing panes of any size in curtain walls and individual interior or exterior panes more than 16 ft^2 (1.5 m^2) in area are laminated annealed or laminated heat-strengthened glass and are detailed to remain in the frame when cracked.	13.6.1.5	A.7.4.8
Masonry Veneer			
C NC N/A U	**HR—not required; LS—LMH; PR—LMH**. TIES: Masonry veneer is connected to the backup with corrosion-resistant ties. There is a minimum of one tie for every 2-2/3 ft^2 (0.25 m^2), and the ties have spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 36 in. (914 mm); for Life Safety in High Seismicity and for Position Retention in any seismicity, 24 in. (610 mm).	13.6.1.2	A.7.5.1
C NC N/A U	**HR—not required; LS—LMH; PR—LMH**. SHELF ANGLES: Masonry veneer is supported by shelf angles or other elements at each floor above the ground floor.	13.6.1.2	A.7.5.2
C NC N/A U	**HR—not required; LS—LMH; PR—LMH**. WEAKENED PLANES: Masonry veneer is anchored to the backup adjacent to weakened planes, such as at the locations of flashing.	13.6.1.2	A.7.5.3
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH**. UNREINFORCED MASONRY BACKUP: There is no unreinforced masonry backup.	13.6.1.1 13.6.1.2	A.7.7.2
C NC N/A U	**HR—not required; LS—MH; PR—MH**. STUD TRACKS: For veneer with cold-formed steel stud backup, stud tracks are fastened to the structure at a spacing equal to or less than 24 in. (610 mm) on center.	13.6.1.1 13.6.1.2	A.7.6.1

continues

Table 17-38 *(Continued)*. Nonstructural Checklist

Status	Evaluation Statement[a,b]	Tier 2 Reference	Commentary Reference
C NC N/A U	**HR—not required; LS—MH; PR—MH**. ANCHORAGE: For veneer with concrete block or masonry backup, the backup is positively anchored to the structure at a horizontal spacing equal to or less than 4 ft along the floors and roof.	13.6.1.1 13.6.1.2	A.7.7.1
C NC N/A U	**HR—not required; LS—not required; PR—MH**. WEEP HOLES: In veneer anchored to stud walls, the veneer has functioning weep holes and base flashing.	13.6.1.2	A.7.5.6
C NC N/A U	**HR—not required; LS—not required; PR—MH**. OPENINGS: For veneer with cold-formed-steel stud backup, steel studs frame window and door openings.	13.6.1.1 13.6.1.2	A.7.6.2
Parapets, Cornices, Ornamentation, and Appendages			
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH**. URM PARAPETS OR CORNICES: Laterally unsupported unreinforced masonry parapets or cornices have height-to-thickness ratios no greater than the following: for Life Safety in Low or Moderate Seismicity, 2.5; for Life Safety in High Seismicity and for Position Retention in any seismicity, 1.5.	13.6.5	A.7.8.1
C NC N/A U	**HR—not required; LS—LMH; PR—LMH**. CANOPIES: Canopies at building exits are anchored to the structure at a spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 10 ft (3.0 m); for Life Safety in High Seismicity and for Position Retention in any seismicity, 6 ft (1.8 m).	13.6.6	A.7.8.2
C NC N/A U	**HR—H; LS—MH; PR—LMH**. CONCRETE PARAPETS: Concrete parapets with height-to-thickness ratios greater than 2.5 have vertical reinforcement.	13.6.5	A.7.8.3
C NC N/A U	**HR—MH; LS—MH; PR—LMH**. APPENDAGES: Cornices, parapets, signs, and other ornamentation or appendages that extend above the highest point of anchorage to the structure or cantilever from components are reinforced and anchored to the structural system at a spacing equal to or less than 6 ft (1.8 m). This evaluation statement item does not apply to parapets or cornices covered by other evaluation statements.	13.6.6	A.7.8.4
Masonry Chimneys			
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH**. URM CHIMNEYS: Unreinforced masonry chimneys extend above the roof surface no more than the following: for Life Safety in Low or Moderate Seismicity, 3 times the least dimension of the chimney; for Life Safety in High Seismicity and for Position Retention in any seismicity, 2 times the least dimension of the chimney.	13.6.7	A.7.9.1
C NC N/A U	**HR—LMH; LS—LMH; PR—LMH**. ANCHORAGE: Masonry chimneys are anchored at each floor level, at the topmost ceiling level, and at the roof.	13.6.7	A.7.9.2
Stairs			
C NC N/A U	**HR—not required; LS—LMH; PR—LMH**. STAIR ENCLOSURES: Hollow-clay tile or unreinforced masonry walls around stair enclosures are restrained out of plane and have height-to-thickness ratios not greater than the following: for Life Safety in Low or Moderate Seismicity, 15-to-1; for Life Safety in High Seismicity and for Position Retention in any seismicity, 12-to-1.	13.6.2 13.6.8	A.7.10.1
C NC N/A U	**HR—not required; LS—LMH; PR—LMH**. STAIR DETAILS: The connection between the stairs and the structure does not rely on post-installed anchors in concrete or masonry, and the stair details are capable of accommodating the drift calculated using the Quick Check procedure of Section 4.4.3.1 for moment-frame structures or 0.5 in. for all other structures without including any lateral stiffness contribution from the stairs.	13.6.8	A.7.10.2
Contents and Furnishings			
C NC N/A U	**HR—LMH; LS—MH; PR—MH**. INDUSTRIAL STORAGE RACKS: Industrial storage racks or pallet racks more than 12 ft high meet the requirements of ANSI/RMI MH 16.1 as modified by ASCE 7, Chapter 15.	13.8.1	A.7.11.1

continues

Table 17-38 *(Continued)*. Nonstructural Checklist

Status	Evaluation Statement[a,b]	Tier 2 Reference	Commentary Reference
C NC N/A U	**HR—not required; LS—H; PR—MH**. TALL NARROW CONTENTS: Contents more than 6 ft (1.8 m) high with a height-to-depth or height-to-width ratio greater than 3-to-1 are anchored to the structure or to each other.	13.8.2	A.7.11.2
C NC N/A U	**HR—not required; LS—H; PR—H**. FALL-PRONE CONTENTS: Equipment, stored items, or other contents weighing more than 20 lb (9.1 kg) whose center of mass is more than 4 ft (1.2 m) above the adjacent floor level are braced or otherwise restrained.	13.8.2	A.7.11.3
C NC N/A U	**HR—not required; LS—not required; PR—MH**. ACCESS FLOORS: Access floors more than 9 in. (229 mm) high are braced.	13.6.10	A.7.11.4
C NC N/A U	**HR—not required; LS—not required; PR—MH**. EQUIPMENT ON ACCESS FLOORS: Equipment and other contents supported by access floor systems are anchored or braced to the structure independent of the access floor.	13.7.7 13.6.10	A.7.11.5
C NC N/A U	**HR—not required; LS—not required; PR—H**. SUSPENDED CONTENTS: Items suspended without lateral bracing are free to swing from or move with the structure from which they are suspended without damaging themselves or adjoining components.	13.8.2	A.7.11.6
Mechanical and Electrical Equipment			
C NC N/A U	**HR—not required; LS—H; PR—H**. FALL-PRONE EQUIPMENT: Equipment weighing more than 20 lb (9.1 kg) whose center of mass is more than 4 ft (1.2 m) above the adjacent floor level, and which is not in-line equipment, is braced.	13.7.1 13.7.7	A.7.12.4
C NC N/A U	**HR—not required; LS—H; PR—H**. IN-LINE EQUIPMENT: Equipment installed in line with a duct or piping system, with an operating weight more than 75 lb (34.0 kg), is supported and laterally braced independent of the duct or piping system.	13.7.1	A.7.12.5
C NC N/A U	**HR—not required; LS—H; PR—MH**. TALL NARROW EQUIPMENT: Equipment more than 6 ft (1.8 m) high with a height-to-depth or height-to-width ratio greater than 3-to-1 is anchored to the floor slab or adjacent structural walls.	13.7.1 13.7.7	A.7.12.6
C NC N/A U	**HR—not required; LS—not required; PR—MH**. MECHANICAL DOORS: Mechanically operated doors are detailed to operate at a story drift ratio of 0.01.	13.6.9	A.7.12.7
C NC N/A U	**HR—not required; LS—not required; PR—H**. SUSPENDED EQUIPMENT: Equipment suspended without lateral bracing is free to swing from or move with the structure from which it is suspended without damaging itself or adjoining components.	13.7.1 13.7.7	A.7.12.8
C NC N/A U	**HR—not required; LS—not required; PR—H**. VIBRATION ISOLATORS: Equipment mounted on vibration isolators is equipped with horizontal restraints or snubbers and with vertical restraints to resist overturning.	13.7.1	A.7.12.9
C NC N/A U	**HR—not required; LS—not required; PR—H**. HEAVY EQUIPMENT: Floor-supported or platform-supported equipment weighing more than 400 lb (181.4 kg) is anchored to the structure.	13.7.1 13.7.7	A.7.12.10
C NC N/A U	**HR—not required; LS—not required; PR—H**. ELECTRICAL EQUIPMENT: Electrical equipment is laterally braced to the structure.	13.7.7	A.7.12.11
C NC N/A U	**HR—not required; LS—not required; PR—H**. CONDUIT COUPLINGS: Conduit greater than 2.5 in. (64 mm) trade size that is attached to panels, cabinets, or other equipment and is subject to relative seismic displacement has flexible couplings or connections.	13.7.8	A.7.12.12
Piping			
C NC N/A U	**HR—not required; LS—not required; PR—H**. FLEXIBLE COUPLINGS: Fluid and gas piping has flexible couplings.	13.7.3 13.7.5	A.7.13.2

continues

Table 17-38 *(Continued)*. Nonstructural Checklist

Status	Evaluation Statement[a,b]	Tier 2 Reference	Commentary Reference
C NC N/A U	**HR—not required; LS—not required; PR—H**. FLUID AND GAS PIPING: Fluid and gas piping is anchored and braced to the structure to limit spills or leaks.	13.7.3 13.7.5	A.7.13.4
C NC N/A U	**HR—not required; LS—not required; PR—H**. C-CLAMPS: One-sided C-clamps that support piping larger than 2.5 in. (64 mm) in diameter are restrained.	13.7.3 13.7.5	A.7.13.5
C NC N/A U	**HR—not required; LS—not required; PR—H**. PIPING CROSSING SEISMIC JOINTS: Piping that crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements.	13.7.3 13.7.5	A.7.13.6
Ducts			
C NC N/A U	**HR—not required; LS—not required; PR—H**. DUCT BRACING: Rectangular ductwork larger than 6 ft^2 (0.56 m^2) in cross-sectional area and round ducts larger than 28 in. (711 mm) in diameter are braced. The maximum spacing of transverse bracing does not exceed 30 ft (9.2 m). The maximum spacing of longitudinal bracing does not exceed 60 ft (18.3 m).	13.7.6	A.7.14.2
C NC N/A U	**HR—not required; LS—not required; PR—H**. DUCT SUPPORT: Ducts are not supported by piping or electrical conduit.	13.7.6	A.7.14.3
C NC N/A U	**HR—not required; LS—not required; PR—H**. DUCTS CROSSING SEISMIC JOINTS: Ducts that cross seismic joints or isolation planes or are connected to independent structures have couplings or other details to accommodate the relative seismic displacements.	13.7.6	A.7.14.4
Elevators			
C NC N/A U	**HR—not required; LS—H; PR—H**. RETAINER GUARDS: Sheaves and drums have cable retainer guards.	13.7.11	A.7.16.1
C NC N/A U	**HR—not required; LS—H; PR—H**. RETAINER PLATE: A retainer plate is present at the top and bottom of both car and counterweight.	13.7.11	A.7.16.2
C NC N/A U	**HR—not required; LS—not required; PR—H**. ELEVATOR EQUIPMENT: Equipment, piping, and other components that are part of the elevator system are anchored.	13.7.11	A.7.16.3
C NC N/A U	**HR—not required; LS—not required; PR—H**. SEISMIC SWITCH: Elevators capable of operating at speeds of 150 ft/min (0.30 m/min) or faster are equipped with seismic switches that meet the requirements of ASME A17.1 or have trigger levels set to 20% of the acceleration of gravity at the base of the structure and 50% of the acceleration of gravity in other locations.	13.7.11	A.7.16.4
C NC N/A U	**HR—not required; LS—not required; PR—H**. SHAFT WALLS: Elevator shaft walls are anchored and reinforced to prevent toppling into the shaft during strong shaking.	13.7.11	A.7.16.5
C NC N/A U	**HR—not required; LS—not required; PR—H**. COUNTERWEIGHT RAILS: All counterweight rails and divider beams are sized in accordance with ASME A17.1.	13.7.11	A.7.16.6
C NC N/A U	**HR—not required; LS—not required; PR—H**. BRACKETS: The brackets that tie the car rails and the counterweight rail to the structure are sized in accordance with ASME A17.1.	13.7.11	A.7.16.7
C NC N/A U	**HR—not required; LS—not required; PR—H**. SPREADER BRACKET: Spreader brackets are not used to resist seismic forces.	13.7.11	A.7.16.8
C NC N/A U	**HR—not required; LS—not required; PR—H**. GO-SLOW ELEVATORS: The building has a go-slow elevator system.	13.7.11	A.7.16.9

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.
[a] Performance Level: HR = Hazards Reduced, LS = Life Safety, and PR = Position Retention.
[b] Level of Seismicity: L = Low, M = Moderate, and H = High.

Concord

1001 Galaxy Way, Suite 310
Concord CA 95420
925-949-5800

Davis

2020 Research Park Drive, Suite 100
Davis CA 95618
530-756-5905

Lake Forest

23692 Birtcher Drive
Lake Forest CA 92630
949-420-3030

Lake Oswego

5 Centerpointe Drive, Suite 130
Lake Oswego OR 97035
503-451-4500

Oceanside

804 Pier View Way, Suite 100
Oceanside CA 92054
760-795-0365

Phoenix

4505 E Chandler Boulevard, Suite 230
Phoenix AZ 85048
602-337-6110

Pleasanton

6800 Koll Center Parkway, Suite 150
Pleasanton CA 94566
925-426-2580

Sacramento

8950 Cal Center Drive, Bldg. 1, Suite 363
Sacramento CA 95826
916-306-2250

Santa Rosa

2235 Mercury Way, Suite 105
Santa Rosa CA 95407
707-543-8506

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