City of Scappoose

WATER SYSTEM MASTER PLAN UPDATE

FINAL | January 2020

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Abbreviations

AACE	American Academy of Cost Engineers
ACI	American Concrete Institute
ADD	average day demand
AL	action level (AL
ASCE	American Society of Civil Engineers
AWWA	American Water Works Association
BPOE	Basic Performance Objectives for Existing Buildings
BPS	booster pump station
BSE-1E	Basic Safety Earthquake 1 for Existing Buildings
oC	degree(s) Celsius
CaCO3	calcium carbonate
Carollo	Carollo Engineers, Inc.
CCI	construction cost index
CDBG	Community Development Block Grant
cf	cubic feet
cfs	cubic feet per second
CI	cast iron
CIP	Capital Improvement Plan
City	City of Scappoose
CSZ	Cascadia Subduction Zone
D	distribution
D/DBPR	Disinfectant/Disinfection By-product Rule
DBP	Disinfection By-product
DEQ	Oregon Department of Environmental Quality
DI	ductile iron
DOGAMI	Department of Geology and Mineral Industries
DSCR	Debt Service Coverage Ratio
DWSP	Drinking Water Source Protection Fund
ENR	Engineering News-Record
EPS	Extended Period Simulation
ERU	Equivalent Residential Unit
ERU/acre	Equivalent Residential Unit per Acre
FF	Fire Flow
ft	feet
ft/s	feet per second
GAC	Granular Activated Carbon
gal	gallon

GIP	galvanized iron pipe
GIS	Geographic Information System
gpd	gallons per day
gpd/ERU	gallons per day per Equivalent Residential Unit
gpd/sf	gallons per day per square foot
gph	gallons per hour
gpm	gallons per minute
gpm/sf	gallons per minute per square foot
GWR	Groundwater Rule
HAA5	Haloacetic acids
HDPE	high-density polyethylene
HGL	Hydraulic Grade Level
HP	Horsepower
HPC	heterotrophic plate counts
ID	Identification
IDSE	Initial Distribution System Evaluation
IFA	Business Oregon – Infrastructure Finance Authority
IO	Immediate Occupancy
IOC	Inorganic Contaminant
kW	kilowatt
lb/day	Pounds per day
LCR	Lead and Copper Rule
LF	Linear Feet
LOS	Level of Service
LRAA	locational running annual averages
LT2ESWTR	Long-Term Stage 2 Enhanced Surface Water Treatment Rule
M	Million
MCL	maximum contaminant level
MDD	Maximum Day Demand
MFL	million fibers per liter
MG	Million Gallons
µg/L	micrograms per liter
mg/L	milligrams per liter
mgd	million gallons per day
Misc	Miscellaneous
MJA	McMillan Jacobs Associates
mL	milliliter
MR1	Miller Road Well #1
MR2	Miller Road Well #2

MR3	Miller Road Well #3
NDWAC	National Drinking Water Advisory Council
NTU	nephelometric turbidity unit
O&M	Operation and Maintenance
OAR	Oregon Administrative Rule
OD	Outer Diameter
OHA	Oregon Health Authority
OMIC	Oregon Manufacturing Innovation Center
ORP	oxidation/reduction potentiometer
ORS	Oregon Revised Statues
PCB	Polychlorinated Biphenyl
pCi/L	picoCuries per liter
PGD	permanent ground deformation
PHD	Peak Hour Demand
Plan	Water System Master Plan Update
POD	Point of Diversion
ppm	parts per million
PRV	Pressure Reducing Valve
PS	Pump Station
psi	pounds per square inch
PVC	polyvinyl chloride
PWS	public water systems
PZ1	Low Zone
PZ2	High Zone
PZ3	Intermediate Zone
PZ4	Dutch Canyon Zone
R&R	Repair and Replace
RAA	running annual averages
ROW	Right-of-way
RUL	remaining useful life
RWSA	Retail Water Service Area
S	supply
SCADA	Supervisory Control and Data Acquisition
SDC	system development charge
SDWA	Safe Drinking Water Act
SDWRLF	Safe Drinking Water Revolving Loan Fund
sf	square feet
SOC	Synthetic Organic Contaminant
ST	storage

SWAP	Source Water Assessment and Protection
T	treatment
TBD	To Be Determined
TCLEE	Technical Committee on Lifelines Earthquake Engineering Monograph
TCR	Total Coliform Rule
TGD	transient ground deformation
TTHM	total Trihalomethanes
UCM	Unregulated Contaminant Monitoring
UCMR1	First Unregulated Contaminant Monitoring Rule
UCMR2	Second Unregulated Contaminant Monitoring Rule
UCMR3	Third Unregulated Contaminant Monitoring Rule
UCMR4	Fourth Unregulated Contaminant Monitoring Rule
UGA	Urban Growth Area
USEPA	U.S. Environmental Protection Agency
VFD	Variable Frequency Drive
VOC	Volatile Organic Contaminant
WMCP	Water Management and Conservation Plan
WRD	Oregon Water Resources Department
WTP	Water Treatment Plant

EXECUTIVE SUMMARY

This Water System Master Plan Update (Plan) updates the City of Scappoose's (City's) 2001 Water System Master Plan Update. The Plan was developed in accordance with Oregon Administrative Rule (OAR) 333-061-0060 (5). It was developed collaboratively by City Staff and Carollo Engineers, Inc. (Carollo). The Plan provides a comprehensive evaluation of the water system. A summary of key aspects is provided below.

ES.1 Existing System

The City currently serves a population of approximately 7,686 people through 2,661 accounts. The existing service area is within the current City limits, as well as the Western Dutch Canyon Service Area and the Raymond Creek Area. The City intends to provide water service to all customers within City limits and its urban growth boundary, including the Oregon Manufacturing Innovation Center and the East Airport development.

The City obtains water supplies from three surface water diversions and four permanent groundwater wells. The City's supply sources are treated at two treatment plants located at Keys Road and Miller Road. The distribution system comprises of two pump stations, five potable water reservoirs with over 3.6 million gallons of storage, three pressure reducing valves, and over 52 miles of transmission and distribution piping ranging from 2-inches to 24-inches in diameter. The City's service area is divided into four pressure zones to provide service customers at varying land elevations.

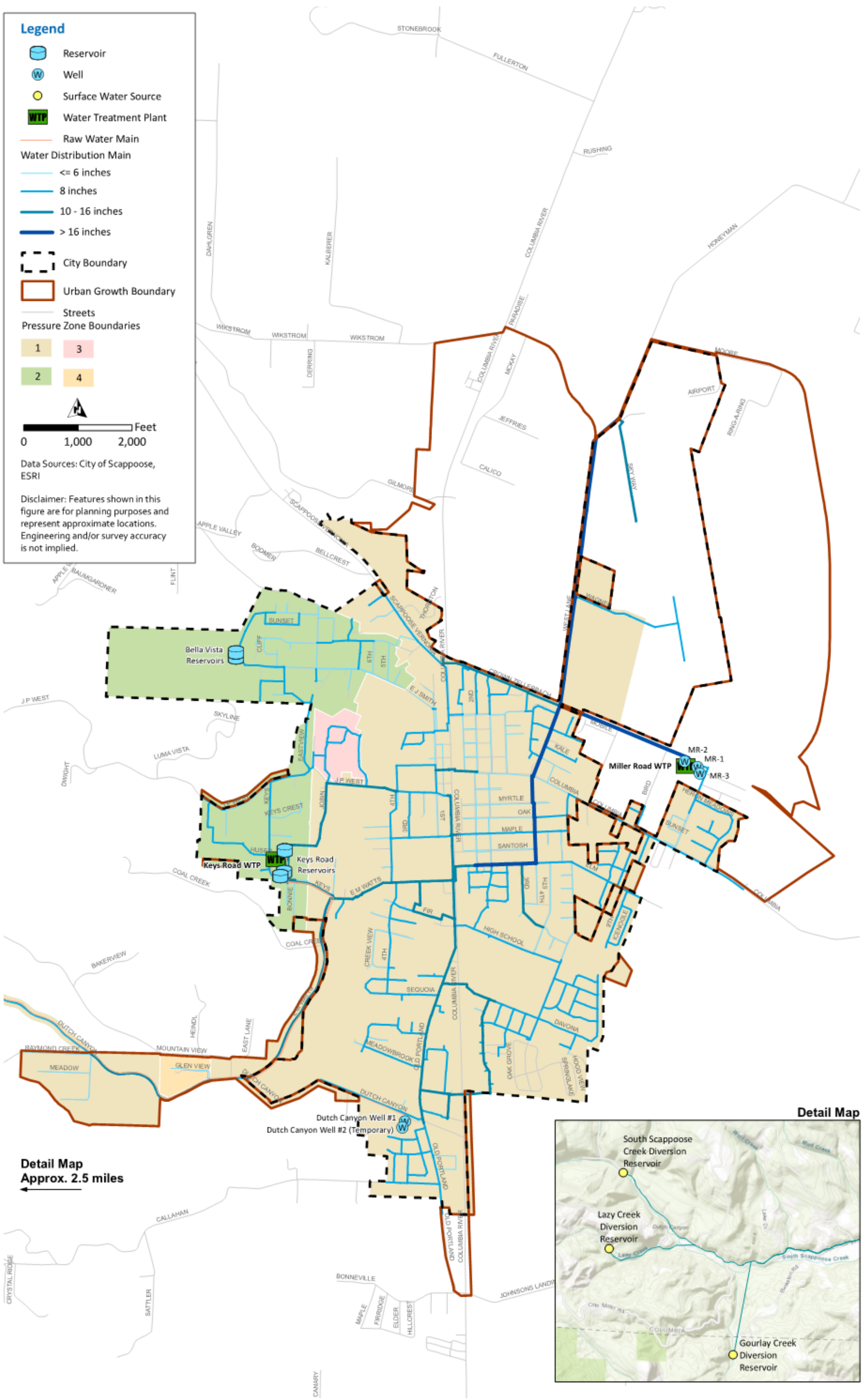


Figure ES.1 City of Scappoose Water System

ES.2 Water Requirements

The City's future water demand for the 20 year planning period was projected using a demographic analysis and historical water production and consumption trends. Projecting realistic future water demands is necessary for evaluating the capability of the water system to meet future water service requirements, planning for infrastructure projects, and securing adequate water supply. Accurate demand projections require a detailed demographic analysis to predict where and how much growth will occur. Growth rates were developed from the City's Transportation System Plan and City projections for the annexation area. Historical production and consumption data were used to estimate future water use rates. Combining these factors, the future maximum day demand (MDD) and average day demand (ADD) were projected. The projected demands are shown in Figure ES.2. The City's water demand is projected to more than double in the next 20 years. To serve these new customers the City will need to develop new supplies, as well as improve the distribution system.

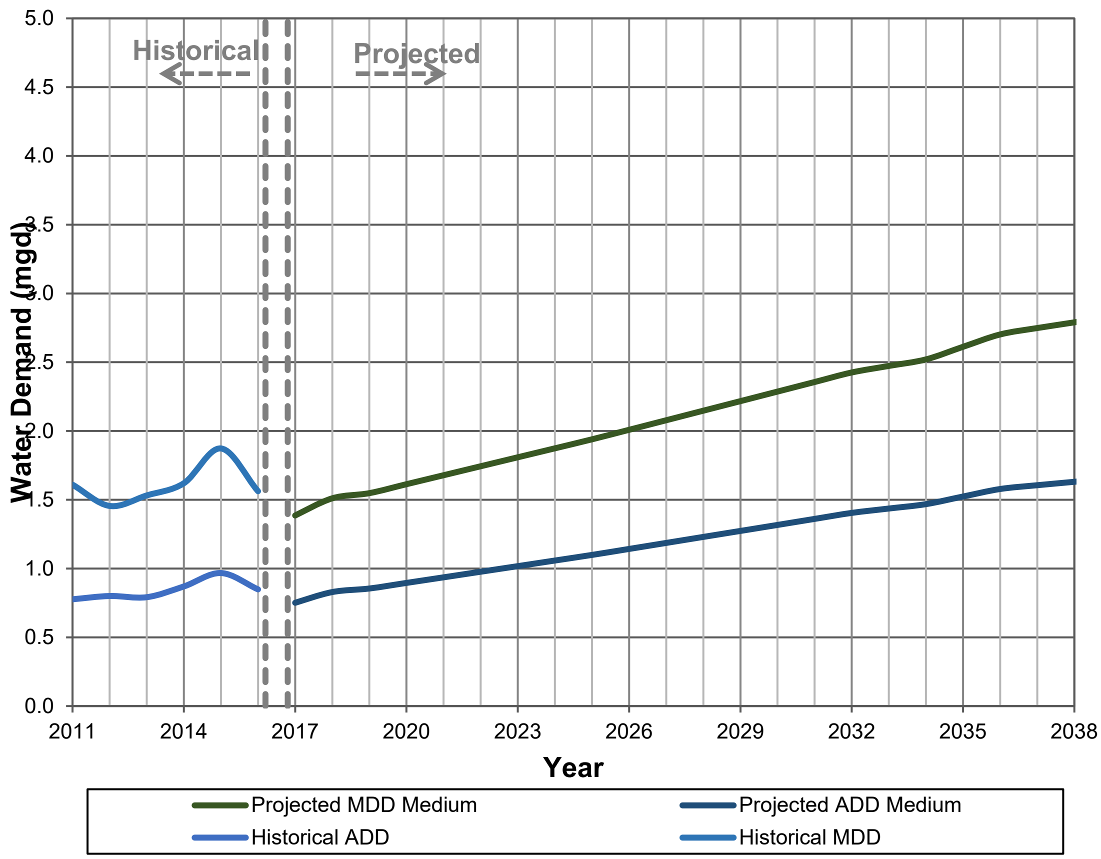


Figure ES.2 Future Water Supply Needs

ES.3 Water Resources

The City's existing water supply sources are a combination of surface water supplies from diversion structures on South Fork Scappoose Creek, Lazy Creek, and Gourlay Creek, and four groundwater wells (Dutch Canyon, Miller Road #1, Miller Road #2, and Miller Road #3). The City holds water rights to withdraw a combined total of 9.0 million gallons per day (mgd) from all three surface water sources. The City's groundwater rights authorize a total withdrawal of 0.94 mgd within the South Scappoose basin and 1.94 mgd within the Jackson Creek Basin.

However, the City's is not able to reliably use its water rights. The City's surface water is limited during the dry season to as little as 0.25 mgd due low stream flows. Groundwater wells are limited based on capacity and currently have water right development limitations that restrict expanded use. In conjunction with the Plan, the City has submitted a Water Management and Conservation Plan for state review to allow full use of their water rights.

To meet the projected demand, the City will need to make full use of their groundwater rights. Additionally, it is anticipated that they will reduce existing water loss, effectively providing 0.13 mgd of supply. A very small amount of new supply will be needed by the end of the 20 year planning period. Three sources of supply were considered feasible: groundwater, Multnomah Channel using a Ranney well, and, to a lesser extent, the existing surface water supply. To better understand the future supply options, it is recommended the City perform investigative activities to better understand the available supply capacity and water quality.

ES.4 Water Quality and Treatment

The City's Water Quality Monitoring Program complies with federal drinking water regulations as adopted by the State of Oregon. The City's water treatment facilities' finished water supply quality meets and/or exceeds all applicable current and future anticipated regulatory requirements. Additionally, the City's distribution system meets all applicable regulations. The City will continue to closely monitor and address changes in regulation to continue to meet all water quality requirements.

ES.5 Distribution System

The distribution system was evaluated to determine if improvements were necessary to meet short-term and long-term future conditions. The distribution system evaluation included pumping capacity and reliability, the capacity of its storage facilities, and for adequate pressures and fire flow (FF) capacity using the City's updated hydraulic model. The City will require approximately 1.5 million gallons (MG) of new storage to meet the projected demand growth. Additionally, the City will require new pipeline upsize and new pipe installation projects which are recommended to ensure required FFs are available to all water mains in the service area.

ES.6 Operations and Maintenance

The City's water system is generally well maintained, but is aging. The operations and maintenance (O&M) evaluation evaluated treatment facilities and the distribution system. Several improvements are recommended to address O&M identified issues at the treatment plants and to reduce the City's high water loss rate. Additionally expanded preventative maintenance and repair and replacement (R&R) are recommended. To complete these activities, it is recommended the City hire two new water operators and create an annual pipeline R&R program focused largely on the aging steel pipes.

ES.7 Capital Improvement Plan

The City's updated Capital Improvement Plan (CIP) was developed based on the improvements identified in the Plan. Conceptual project costs were developed for budgeting purposes. These American Academy of Cost Engineers (AACE) Class IV estimates are anticipated to have an accuracy range of +50 percent and -30 percent. Short-term projects were prioritized and assigned to individual years from 2019 through 2028. Projects in the long-term, after 2028, are not scheduled to individual years due to future uncertainties and are shown as a single combined long-term total. Table ES.3 provides a breakdown of the CIP by project. The total short-term CIP cost is $18,786,000, or $1,708,000 per year. Long-term CIP cost is $36,340,000, or $3,634,000 per year. Additional project details can be found in Chapter 8.

Table ES.1 Summary of CIP Projects

Project	Cost Type: Current Dollars	Total CIP Cost Estimate	CIP Phasing (Current Dollars)													Developer Share (%)
			2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	Short-term (2018-2028)	Long-term (2029-2038)	
Supply		**$19,500,000**	**$480,000**	**$0**	**$450,000**	**$390,000**	**$1,580,000**	**$420,000**	**$1,680,000**	**$420,000**	**$1,680,000**	**$0**	**$0**	**$7,100,000**	**$12,400,000**	
S-01	Dutch Canyon Well #2	$480,000	$480,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$480,000	$0	0%
S-02	Dutch Canyon Well #3	$2,100,000	$0	$0	$0	$0	$0	$420,000	$1,680,000	$0	$0	$0	$0	$2,100,000	$0	0%
S-03	Miller Road Well #4	$200,000	$0	$0	$200,000	$0	$0	$0	$0	$0	$0	$0	$0	$200,000	$0	100%
S-04	Miller Road Well #5	$1,970,000	$0	$0	$0	$390,000	$1,580,000	$0	$0	$0	$0	$0	$0	$1,970,000	$0	0%
S-05	Miller Road Well #6	$2,100,000	$0	$0	$0	$0	$0	$0	$0	$420,000	$1,680,000	$0	$0	$2,100,000	$0	0%
S-06	Long-Term Supply	$12,650,000	$0	$250,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$250,000	$12,400,000	0%
Treatment		**$1,460,000**	**$0**	**$410,000**	**$100,000**	**$0**	**$0**	**$600,000**	**$350,000**	**$0**	**$0**	**$0**	**$0**	**$1,460,000**	**$0**	
T-01	Miller Road R&R	$650,000	$0	$150,000	$0	$0	$0	$500,000	$0	$0	$0	$0	$0	$650,000	$0	0%
T-02	Keys Road R&R	$340,000	$0	$240,000	$100,000	$0	$0	$0	$0	$0	$0	$0	$0	$340,000	$0	0%
T-03	Supply and Treatment Plant LOS Goals	$20,000	$0	$20,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$20,000	$0	0%
T-04	Seismic and Life-Safety Audit	$160,000	$0	$0	$0	$0	$0	$60,000	$100,000	$0	$0	$0	$0	$160,000	$0	0%
T-05	Treatment Capacity and Operations Optimization Study	$290,000	$0	$0	$0	$0	$0	$40,000	$250,000	$0	$0	$0	$0	$290,000	$0	0%
Distribution		**$22,860,000**	**$0**	**$60,000**	**$60,000**	**$60,000**	**$60,000**	**$110,000**	**$310,000**	**$810,000**	**$60,000**	**$60,000**	**$60,000**	**$1,650,000**	**$21,210,000**	
D-01	NW Eastview Drive Replacement	$790,000	$0	$0	$0	$0	$0	$0	$200,000	$590,000	$0	$0	$0	$790,000	$0	0%
D-02	SW 5th Street Connection	$210,000	$0	$0	$0	$0	$0	$0	$50,000	$160,000	$0	$0	$0	$210,000	$0	0%
D-03	Sky Way Drive Connection Airport Annex	$50,000	$0	$0	$0	$0	$0	$50,000	$0	$0	$0	$0	$0	$50,000	$0	100%
D-04	Dutch Canyon Rd to Em Watts Rd	$540,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$540,000	0%
D-05	Moore Rd Airport Annex	$1,630,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$1,630,000	0%
D-06	Airport Annex North of Bird Rd	$610,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$610,000	0%
D-07	Water Main Repair and Replacement	$15,500,000	$0	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$500,000	$15,000,000	0%
D-08	Dead-End and Small Diameter Mains	$3,530,000	$0	$10,000	$10,000	$10,000	$10,000	$10,000	$10,000	$10,000	$10,000	$10,000	$10,000	$100,000	$3,430,000	0%
Pump Stations		**$480,000**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$480,000**	
PS-01	High Zone BPS	$480,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$480,000	0%
Storage		**$5,256,000**	**$0**	**$0**	**$0**	**$900,000**	**$0**	**$0**	**$0**	**$0**	**$0**	**$1,089,000**	**$3,267,000**	**$5,256,000**	**$0**	
ST-01	2.0 MG Keys Road Reservoir	$4,356,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$1,089,000	$3,267,000	$4,356,000	$0	0%
ST-02	Reservoir Seismic Retrofit	$900,000	$0	$0	$0	$900,000	$0	$0	$0	$0	$0	$0	$0	$900,000	$0	0%
Miscellaneous		**$5,570,000**	**$585,000**	**$485,000**	**$250,000**	**$250,000**	**$250,000**	**$250,000**	**$250,000**	**$250,000**	**$250,000**	**$250,000**	**$250,000**	**$3,320,000**	**$2,250,000**	
Misc-01	City's Capital Outlay Projects	$5,570,000	$585,000	$485,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$3,320,000	$2,250,000	
CIP Total (Current Dollars)		**$55,126,000**	**$1,065,000**	**$955,000**	**$860,000**	**$1,600,000**	**$1,890,000**	**$1,380,000**	**$2,590,000**	**$1,480,000**	**$1,990,000**	**$1,399,000**	**$3,577,000**	**$18,789,000**	**$36,340,000**	
Annual Cost (Current Dollars)		**$2,625,000**	**$1,065,000**	**$955,000**	**$860,000**	**$1,600,000**	**$1,890,000**	**$1,380,000**	**$2,590,000**	**$1,480,000**	**$1,990,000**	**$1,399,000**	**$3,577,000**	**$1,708,000**	**$3,634,000**	

ES.8 Financial

An analysis of the financial status of the water utility was conducted to determine the impact of the new CIP on the City water utility finances, which is described in Chapter 8. The analysis found the City would need to raise rates, obtain grants, and/or take on debt to finance the CIP. If the City chooses not to issue any debt, water rates are anticipated to increase 5.5 percent per year for the next ten years. The City also has the option to issue debt, which would lower the annual rate increases over the next ten years to 3.0 percent. The City can also apply for qualifying grants and low interest loans to reduce the overall CIP costs. It is recommended to conduct a rate and cost of service study to determine the appropriate rate increases for each customer class, which would help cover the costs of the projected capital projects. It is also recommended to complete a system development charge (SDC) study to confirm the projected charges are adequate for new development.

ES.9 Seismic Mitigation Plan

The City developed a 50 year seismic mitigation plan in accordance with recent Oregon regulations. The mitigation plan, presented in Chapter 10, identified seismic hazards, determined conceptual system performance during a major earthquake, and recommended actions for the City to begin planning for mitigating the expected damage. Consistent with the Plan, the City will mitigate seismic deficiencies at critical facilities in the next 20 years, including:

- Miller Road Water Treatment Plant (WTP) had no significant deficiencies. Planned R&R will address minor noted seismic issues.
- Keys Road WTP Seismic and Lift Safety Audit is planned. The analysis found there is a potential for catastrophic failure of the WTP in a seismic event; a detailed audit is needed to determine next steps focused on Staff safety.
- Reservoir Seismic Retrofits. In the next 20 years, seismic retrofits to mitigate current deficiencies are planned for the Keys Road 1.0 MG reservoir and, if necessary, the Keys Road 2.0 MG reservoir and Bella Vista 2 Reservoir.
- Seismic resilient piping will connect these critical facilities, referred to as a seismic backbone. No specific projects are planned to complete entire sections of the backbone for the next 20 years. However, any pipe installed along the backbone (due to development, condition related replacement, etc.) will be seismic resilient piping.

Additional mitigation of the remaining critical infrastructure will be addressed outside of the 20 year CIP planning period.

Chapter 1

INTRODUCTION

1.1 Purpose

This Water System Master Plan Update (Plan) updates the City of Scappoose's (City's) 2001 Water System Master Plan Update. The Plan was developed in accordance with Oregon Administrative Rule (OAR) 333-061-0060 (5). It was developed collaboratively by City Staff and Carollo Engineers, Inc. (Carollo).

1.2 Authorization and Adoption

In 2017, the City authorized Carollo to prepare this document in accordance with City policies and procedures and all applicable Oregon State rules and regulations. A consumer meeting was publicly advertised and conducted before the Council meeting to adopt this Plan. The Adopting Resolution and Ordinance will be provided in Appendix C.

1.2.1 Related Plans

The Plan incorporates information from the City's related planning efforts that include:

- City of Scappoose Water Management and Conservation Plan, 2012, updated 2018.
- City of Scappoose Comprehensive Plan, 2001, amended 2018.

1.2.2 Comments and Responses from Agencies and Adjacent Purveyors

The draft Plan was sent to Columbia County and the Oregon Health Authority (OHA) for review. Comments and responses will be included in Appendix D.

1.3 Plan Organization

This Plan contains ten chapters, followed by appendices that provide supporting documentation for the information presented in the report. The chapters of the Plan are organized as follows:

Chapter 1 – Introduction summarizes the Plan purpose and a reference index to regulatory required information.

Chapter 2 – Existing System describes the existing water system.

Chapter 3 – Water Requirements presents the methodology for developing future water demands and the existing and future water demands.

Chapter 4 – Water Resources reviews the City's existing water rights and supplies and identifies future alternatives for future supply sources.

Chapter 5 – Water Quality and Treatment assesses the City's water quality, current and anticipated water quality regulations and compliance, treatment plant performance, and recommendations for treatment plant repair and replacement improvements.

Chapter 6 – Water System Analysis evaluates the water distribution system for its ability to meet reliability criteria under future demands and provides recommendations to eliminate identified system deficiencies.

Chapter 7 – Operations and Maintenance details operation and maintenance requirements for the water system including staffing needs for preventative maintenance, water loss mitigation efforts, and pipeline repair and replacement programs.

Chapter 8 – Capital Improvement Plan summarizes the recommended capital improvement projects to address identified system deficiencies, including project costs and scheduling.

Chapter 9 – Financial Plan details financing strategies to fund the recommended capital improvement plan.

Chapter 10 – Seismic Mitigation Plan presents a risk assessment and mitigation plan for the water system.

1.4 Plan Index for Regulatory Requirements

Table 1.1 summarizes plan requirements and identifies the location of the pertinent information.

Table 1.1 OAR 333-061-0060 (5) Specific Requirements

	Item	Location
1.	Summary of the overall plan including:	
a.	Water quality and service goals.	Executive Summary
b.	Present and future water system deficiencies.	Executive Summary
c.	The engineer's recommended alternative for achieving the goals and correcting the deficiencies.	Executive Summary
d.	The recommended implementation schedule.	Executive Summary
e.	A financing program for constructing the improvements.	Executive Summary
2.	A description of the existing water system including:	
a.	The service area.	Chapter 2
b.	The source(s) of supply.	Chapter 2
c.	Status of water rights.	Chapter 4
d.	Current status of drinking water quality and compliance with regulatory standards.	Chapter 5
e.	Maps or schematics of the water system showing size and location of facilities.	Chapter 2
f.	Operation and maintenance requirements.	Chapter 7
3.	A description of water quality and level of service goals for the water system, considering, as appropriate:	
a.	Existing and future (near term) regulatory requirements.	Chapter 5
b.	Non-regulatory water quality needs of water users.	Chapter 5
c.	Flow and pressure requirements.	Chapter 6
d.	Capacity needs related to water use and fire flow needs.	Chapter 6

Table 1.1 OAR 333-061-0060 (5) Specific Requirements (continued)

Item		Location
4.	An estimate of the projected growth of the water system during the master plan period and the impacts on:	
a.	The service area boundaries.	Chapter 3
b.	The water supply source(s) and availability.	Chapter 3
c.	Customer water use.	Chapter 3
5.	An engineering evaluation of the ability of the existing water system to meet the water quality and service goals, identification of existing water system deficiencies, and deficiencies likely to develop within the master plan period. The evaluation shall include:	
a.	The water supply source.	Chapter 4
b.	Water treatment, storage and distribution facilities.	Chapter 5, Chapter 6
c.	O&M requirements.	Chapter 5, Chapter 7
d.	A description of the water rights with a determination	Chapter 4
e.	The impacts of present and probable future drinking water quality regulations.	Chapter 5
6.	The master plan shall include an identification of the following which may be needed to correct water system deficiencies and achieve system expansion to meet anticipated growth:	
a.	Alternative engineering solutions.	Chapter 4, Chapter 5, Chapter 6
b.	Environmental impacts.	
c.	Associated capital costs.	Chapter 4, Chapter 6, Chapter 8
d.	Operational and maintenance costs.	Chapter 7
e.	Identification of available options for cooperative or coordinated water system improvements with other local water suppliers.	Chapter 4
7.	A description of alternatives to finance water system improvements including local financing (such as user rates and system development charges) and financing assistance programs.	Chapter 9
8.	A recommended water system improvement program including:	
a.	The recommended engineering alternative and associated costs.	Chapter 8
b.	Maps or schematics showing size and location of proposed facilities.	Chapter 8
c.	The recommended financing alternative.	Chapter 9
d.	A recommended schedule for water system design and construction.	Chapter 8
9.	If required as a condition of a water use permit issued by the Oregon Water Resources Department, the master plan shall address the requirements of OAR 690-086-0120 (Water Management and Conservation Plans).	A Water Management and Conservation Plan has been submitted separately

Table 1.1 OAR 333-061-0060 (5) Specific Requirements (continued)

Item	Location
10. 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 heavy damage potential using the Map of Earthquake and Tsunami Damage Potential for 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:	
a. 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.	Chapter 10
b. The seismic risk assessment must identify and evaluate the likelihood and consequences of seismic failures for each critical facility.	Chapter 10
c. The mitigation plan may encompass a 50-year planning horizon and include recommendations to minimize water loss for each critical facility, capital improvements or recommendations for further study or analysis.	Chapter 10

Chapter 2

EXISTING SYSTEM

2.1 Service Area

The City of Scappoose (City) currently serves a population of approximately 7,686 people through 2,661 accounts. The existing service area is within the current City limits with the exception of the following two areas; Western Dutch Canyon Service Area and the Raymond Creek Area. The City intends to provide water service to all customers, within its Urban Growth Boundary in the future, as required. The City is annexing portions of its northern Urban Growth Boundary area, which will aid in the development of the Oregon Manufacturing Innovation Center and the East Airport development. The City currently has 169 acres zoned for commercial use of which 42 acres are undeveloped. Similarly, there are 86 acres zoned for industrial use, with 49 acres currently undeveloped. In addition, there are 411 acres zoned as public use airport (which allows a mix of commercial and light industrial use), with 361 acres undeveloped.

2.2 Sources of Supply

The City obtains water supplies from three surface water diversions and four permanent groundwater wells. Surface water sources are on the South Scappoose Creek and its tributaries, Gourlay Creek and Lazy Creek. Raw surface water is combined into a single transmission main and conveyed to the Keys Road water treatment facility. Groundwater wells are located in two different well fields: Dutch Canyon and Miller Road.

2.2.1 System Operations

The City's supply sources are treated at two treatment plants located at Keys Road and Miller Road. The combined surface sources are conveyed to the Keys Road treatment plant through approximately 4.7 miles of 12 3/4-inch and 1.6 miles of 8-inch tar wrapped steel pipe. The Dutch Canyon well water is pumped by a 50 horsepower (HP) vertical turbine pump through 1.7 miles of 12-inch ductile iron (DI) and polyvinyl chloride (PVC) pipe. The Miller Road wells are pumped directly into the treatment plant for on-site treatment.

2.3 Existing Facilities

The Scappoose water system comprises of two treatment plants (Keys Road and Miller Road treatment plants), three surface sources, four wells, two pump stations, five reservoirs, three pressure reducing valves (PRV), and over 52 miles of transmission and distribution piping. The following sections provide a brief description of these system components.

2.3.1 Treatment Facilities

There are three filtration units located at the Keys Road treatment plant. The aluminum package plant was built in 1979 and consists of two mixed media filter trains that are designed primarily for turbidity removal as required for the treatment of surface water. The third filtration unit, built in 2000, is a concrete tank filled with green sand that is used for iron removal from the

Dutch Canyon well. The Miller Road treatment plant consists of two more concrete green sand filters, built in 2004, that are used for iron removal from the three on-site wells.

2.3.2 Reservoirs

There are three concrete reservoirs located at the Keys Road treatment facility; a 2 million gallon (MG) built in 2004, a 1 MG built in 1967, and a 0.2 MG built in 1947. The 0.2 MG reservoir is currently not being used pending seismic evaluation. There are two steel reservoirs located at the Bella Vista site; a 0.30 MG built in 1967 and a 0.37 MG built in 2003. These reservoirs serve the higher elevation customers on the western side of the City.

2.3.3 Pump Stations

The City has two pump stations. The high zone pump station is located at the Keys treatment facility and is dedicated to water transfer from the Keys reservoirs to the Bella Vista reservoirs in order to meet fire flow and pressure needs of higher elevation residents located on the western side of town. The Glen View pump station is required to meet the flow and pressure needs (not fire suppression) of the homes located on Glen View Lane, Mountain View Road, and further up Dutch Canyon Road.

2.3.4 Pressure Zones

The City's service area has 4 pressure zones. Pressure zone 1 (PZ1) serves the majority of the residential, commercial, and some industrial properties. Pressure zone 2 (PZ2) is the high-pressure zone in the Northwest end of the City, serving primarily residential customers, and some industrial users. Pressure zone 3 (PZ3) is a sub-pressure zone connected to PZ2 via a PRV serving residential customers and Veterans Park. Pressure zone 4 (PZ4) is located in the Dutch Canyon service area and provides domestic use only (no fire suppression) supply only to residential customers on Glen View and those further up Dutch Canyon Road.

2.3.5 Pipes

The transmission and distribution system include 52 miles of pipe ranging from 2-inches to 24-inches in diameter. The transmission piping conveys raw water from the surface source to the Keys Road treatment plant and consists of 4.7 miles of 12 3/4-inch steel pipe installed in 1955 and 1.6 miles of 8-inch steel pipe installed in 1967. The 45.7 miles of distribution piping delivers treated water to City's customers. Sixty five percent of the distribution system's pipe material is PVC with 22 percent of the system consisting of older steel pipe and some cast iron pipe. The remainder is DI, galvanized iron, or high-density polyethylene (HDPE).

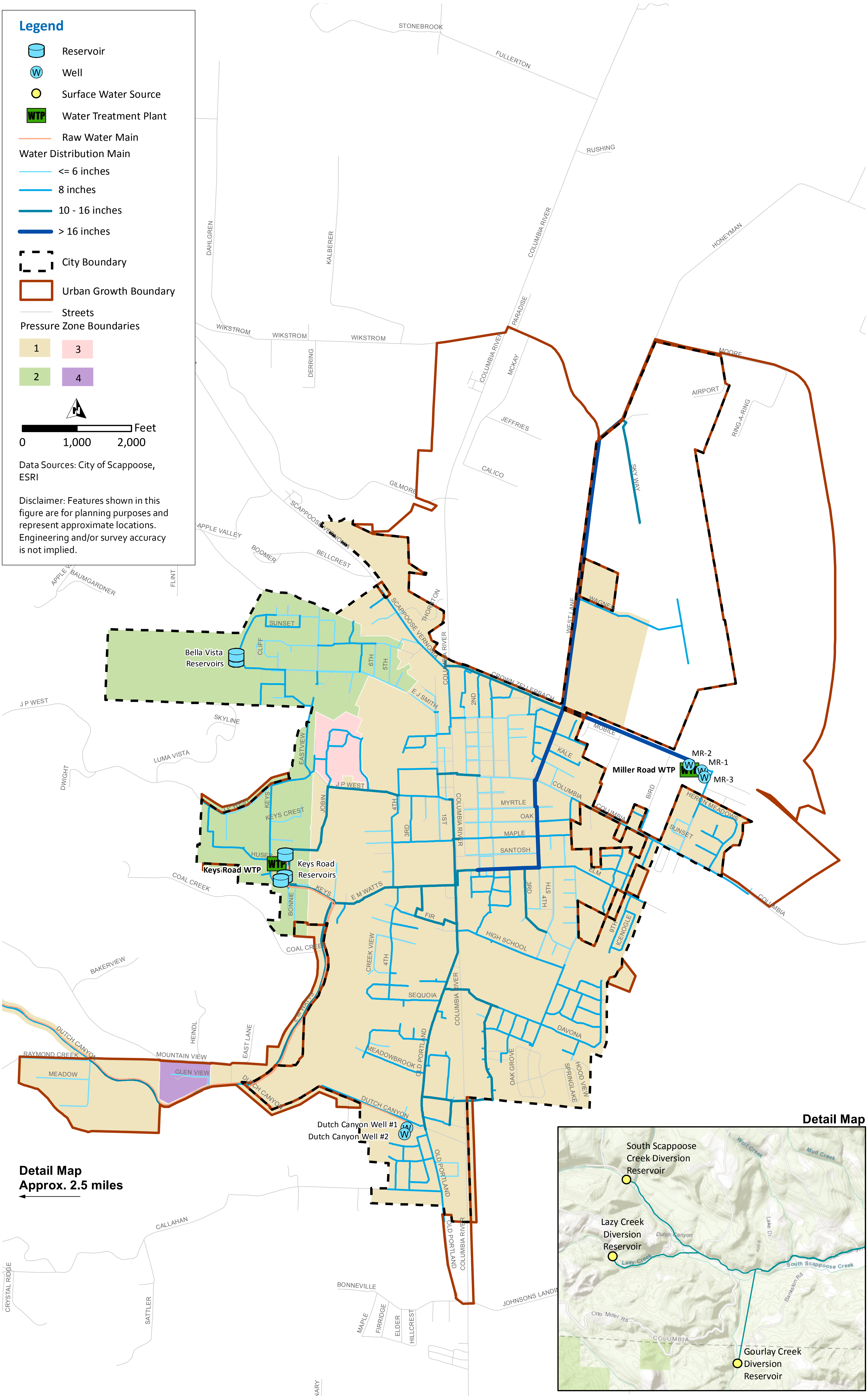


Figure 2.1 City of Scappoose Water System

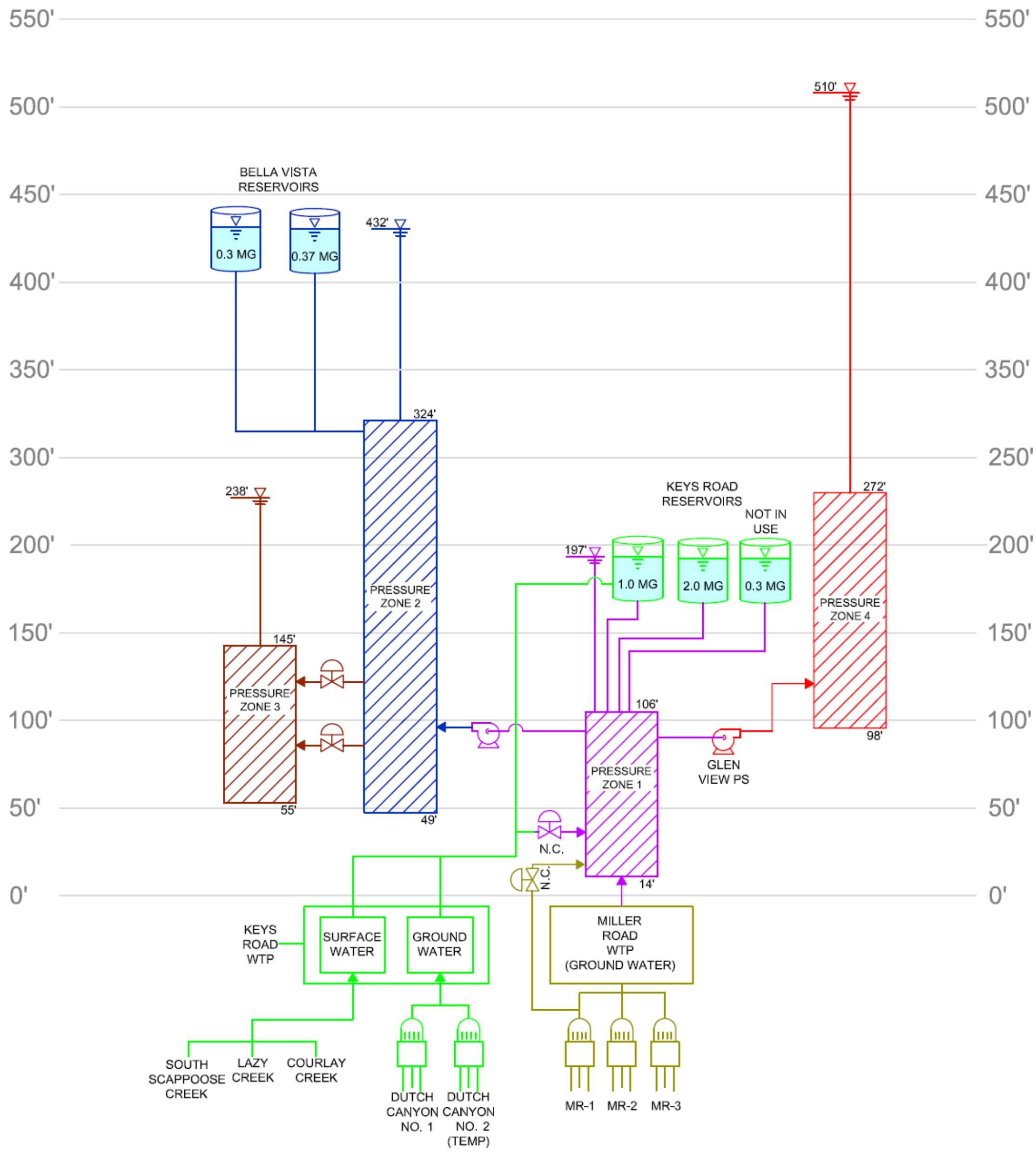


Figure 2.2 Hydraulic Profile

Chapter 3

WATER REQUIREMENTS

3.1 Introduction

This chapter presents a demographic analysis, historical water production and consumption trends, as well as water demand forecasts for the ten and twenty year planning periods for the City of Scappoose's (City's) water system. Projecting realistic future water demands is necessary for evaluating the capability of the water system to meet future water service requirements, planning for infrastructure projects, and securing adequate water supply. Future water demands are used as input conditions for the analyses of the water system that are used to develop the Capital Improvement Plan (CIP).

Accurate demand projections require a detailed demographic analysis to predict where and how much growth will occur. This chapter first describes the existing and future land use within the City. Demographic trends include the City and its proposed annexation areas. Growth rates were developed from the City's Transportation System Plan and City projections for the annexation area.

The next requirement for developing accurate demand projections is a thorough review of the City's unique historical water consumption trends. The unique consumption trends of the City's various customer classes are evaluated from historical customer billing data. The historical average water use for residential customers establishes the City's current Equivalent Residential Unit (ERU) water use. Multiunit residential and non-residential customer consumption is expressed in terms of ERUs based on the comparison of these customers' water use to the ERU value. Historical production data is used to determine the maximum day demand (MDD) to average day demand (ADD) peaking factor. A comparison of historical production data versus historical consumption data determines system water losses.

The water use parameters found in the historical production and consumption data, along with the growth rates developed in the demographic analysis, are used to predict a range of future water demands. Low, medium, and high demand projection scenarios were developed as described in the following sections. The City's water system will be evaluated for capacity deficiencies in the water system analysis based on the medium demand projections presented herein.

Note, summary statistics in this Chapter include the average and 75th percentile. The average represents the typical value over the period of record and, by its nature, is commonly exceeded. The 75th percentile represents the value below which 75 percent of years may be found. For Master Planning, where the goal is to identify needed system improvements, using the 75th percentile value provides a more conservative estimate than using the average values for sizing of infrastructure capacity needs.

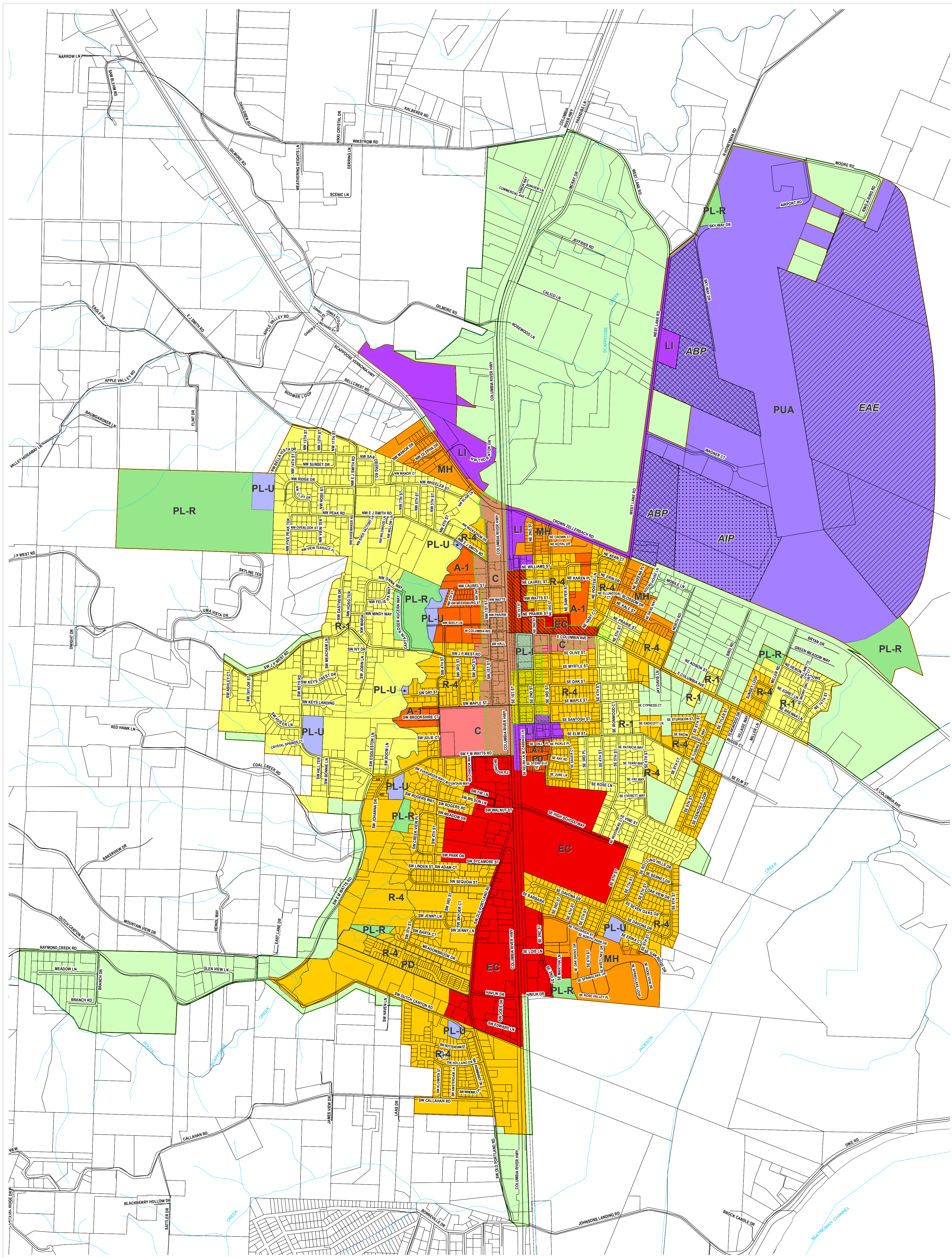
The following sections describe the available data sources, methodologies used, recommended planning values, and resulting demand projections in further detail.

3.2 Land Use

The City's water system currently serves City residents. The City breaks down the existing service area into land use categories, as shown on Figure 3.1.

For planning purposes, the future service area considers supplying additional annexation surrounding the airport. Future land use, shown in Figure 3.2, portrays projected land use in the year 2036. These land use maps are used to distribute existing and future water demands throughout each of the City's pressure zones in the water system hydraulic model for the purpose of evaluating the water distribution system.

Note, existing and future land use designations are consistent with the City's Comprehensive Plan.



City of Scappoose

Figure 3.1 Zoning Map

Legend

- Streets
- Rivers
- Railroads
- Taxlots
- City Limits
- Urban Growth Boundary
- Downtown Overlay
- Airport Business Park Overlay
- Airport Industrial Park Overlay
- East Airport Employment Overlay

Zoning

- R-1 - Low Density Residential
- R-4 - Moderate Density Residential
- MH - Manufactured Housing Residential
- A-1 - High Density Residential
- C - General Commercial
- EC - Expanded Commercial
- LI - Light Industrial
- PUA - Public Use Airport
- PL-R - Public Lands-Recreation
- PL-U - Public Lands-Utility
- PL-I, Public Lands-Institutional

1" = 1500'

0 500 1,000 2,000 Feet

Taxlot Revision: 6/2017

Compilation Date: 7/2017

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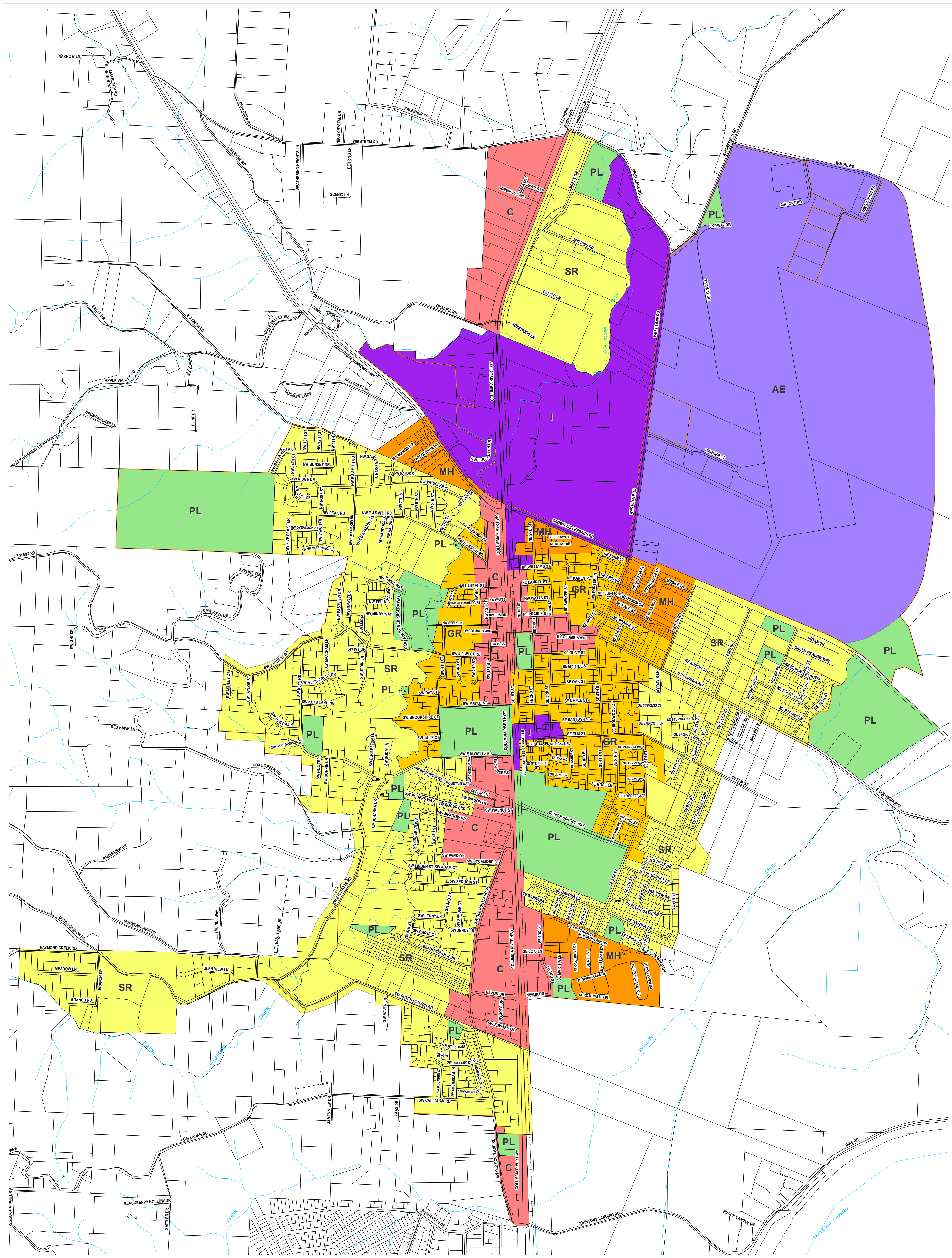

City of Scappoose

Figure 3.2 Comprehensive Plan Map

Legend

- Streets
- Rivers
- Railroads
- Taxlots
- City Limits

Comprehensive Plan Designation

- SR - Suburban Residential
- GR - General Residential
- MH - Manufactured Home
- C - Commercial
- I - Industrial
- AE - Airport Employment
- PL - Public Lands

1" = 1500'

0 500 1,000 2,000 Feet

Taxlot Revision: 6/2017

Compilation Date: 7/2017

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3.3 Demographic Analysis

A demographic analysis was performed for the City's retail water service area (RWSA). The RWSA boundary coincides with the City's urban growth area (UGA) that includes both the City and potential annexation areas. Demographic analyses for the City limits, and areas of the UGA with existing water service, were based on the City's population and employment growth forecasts, as documented in the City's Transportation System Plan. The annual growth rates for population and employee growth are provided in Table 3.1.

In order to predict the City's future water demand, the population growth forecasts were used to calculate residential growth rates and the employment forecasts were used to project non-residential growth rates. During demand projection development, the City's existing number of water accounts is grown by these annual growth rates to predict future number of accounts within the 20 year planning period.

Demographic and demand growth for potential annexation areas was made separately using the City's understanding of proposed developments, as presented in Section 3.5.2.

Table 3.1 Demographic Growth Rates

	Current (2016)	Future (2038)	Annual Growth (percent)
Households	7,560	10,935	1.7
Jobs	2,210	4,520	3.4

3.4 Historical Production and Consumption

Historical production, number of accounts, and consumption data were obtained from City records for the years 2011 through 2016. These historical demand data were evaluated to characterize the unique water use of the City's customers. Several key demand parameters were generated from this data: water use per customer class, water use per connection for each customer class, water use per ERU, MDD to ADD peaking factors, and water losses. These parameters were used as the basis for future demand projections.

3.4.1 Historical Water Production

The City's water system is supplied by four different wells and three surface water supplies. Three of the wells, Miller Road #1 (MR1), Miller Road #2 (MR2), and Miller Road #3 (MR3), pump water to the Miller Road Water Treatment Plant. Surface water from the South Fork Scappoose Creek, Gourlay Creek, and Lazy Creek is treated at the Keys Road Water Treatment Plant along with water from the fourth well, Dutch Canyon No. 1 Well. Water production varies annually in response to system demand, which is correlated to weather, development, economic conditions, and conservation activities.

Surface water supplies have generally been used as peaking supply during the summer months, as well as shoulder months. Water quality concerns have limited use of the surface water supply during the fall and early winter months. Groundwater supplies are used year-round by the City.

Raw and treated water production data are summarized below. Due to supply limitations, the City has been required in the past to use storage, in addition to its supplies, to meet peak demands; therefore, supply from greater than a 24 hour period may have been used to meet the peak demand. ADD and MDD values were based on treated water that enters the system, which measured water leaving the Keys Treatment Plant storage tanks and Miller Road Treatment Plant.

3.4.1.1 Raw Water Production

Raw water production for the period of record are shown in Table 3.2. Raw water production is total metered source water production entering the treatment plant, and treated water production represents metered water entering the distribution system.

Total raw water production, shown in Figure 3.3 by source, increased in 2014 after 3 years of essential unchanged production. Production by source has varied year-to-year, but the majority of the water produced, 67 percent between 2011 and 2016, has come from the four groundwater wells.

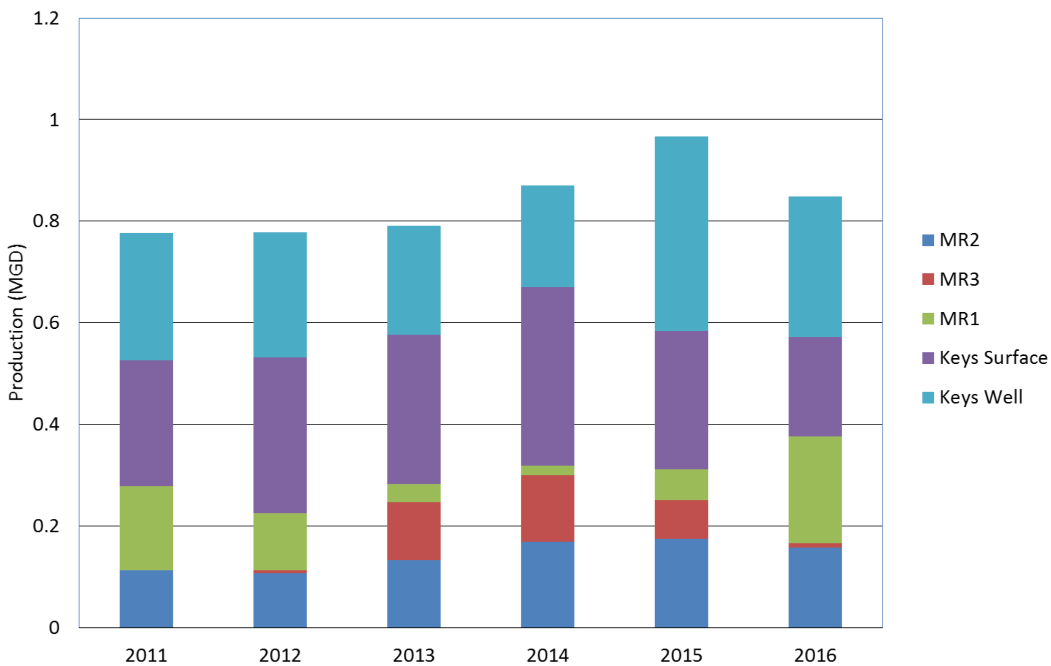


Figure 3.3 Historical Raw Water Production by Source

3.4.1.2 Average Day Demand

The ADD represents the average daily demand for treated water over a year. It is calculated by dividing the total treated water produced by the number of days per year. These values for the years 2011 through 2016 are presented in Table 3.2. The relatively steady water production results in little difference between the summary statistics (e.g. average and 75th percentile). The maximum ADD of 0.79 million gallons per day (mgd) occurred in 2015, which corresponds to a relatively hot and dry summer.

3.4.1.3 Maximum Day Demand

Identifying MDD is critical for establishing system supply capacity, pump station discharge rates, reservoir capacity, and pump sizes. Historical MDD values are equivalent to the highest treated water production in one day in a given year, and are usually during the summer when irrigation is occurring. The historical MDD and date of occurrence for 2011 through 2016 are presented in Table 3.2. MDD has also been consistent over the period analyzed, with a maximum of 1.59 mgd in 2011.

In order to develop future MDD projections, the historical MDD to ADD peaking factor was calculated, as shown in Table 3.2. Peaking factors were based on treated water production to represent all system demands, including authorized use, unauthorized use, and estimated losses. Consistent ADD and MDD values have resulted in consistent peaking factors, save for the maximum of 2.23 in 2011.

Table 3.2 Historical Water Use

Year	Raw Water		Treated Water			
	Average Production (mgd)	Maximum Production (mgd)	Average Day Demand (mgd)	Maximum Day Demand (mgd)	Date of Maximum Day Demand	Max Day/Avg. Day Peaking Factor
2011	0.78	1.61	0.71	1.59	8/2/2011	2.23
2012	0.80	1.46	0.73	1.38	9/19/2012	1.88
2013	0.79	1.53	0.72	1.27	8/16/2013	1.78
2014	0.87	1.62	0.73	1.31	7/30/2014	1.79
2015	0.97	1.87	0.79	1.37	8/1/2015	1.72
2016	0.85	1.56	0.75	1.34	6/4/2016	1.78
Average	0.84	1.61	0.73	1.38		1.86
75th Percentile	0.86	1.62	0.74	1.38		1.86

3.4.2 Historical Customer Accounts

For analysis of water consumption, the City divided its customers into 5 categories as follows:

1. General Residential.
2. Commercial.
3. Industrial.
4. Manufactured Home.
5. Public Lands.

The number of accounts in each customer category for the years 2011 through 2016 is summarized in Table 3.3. The City has a net increase of 137 accounts, about 1 percent per year, during the period of record. Account growth has mainly been residential, through the increase in General Residential accounts and the new Manufactured Home designation created in 2015. Public lands represent City parks, schools, and other facilities.

Table 3.3 Historical Number of Connections

Year	General Residential	Commercial	Industrial	Manufactured Homes	Public Lands	Total
2011	2,087	88	9	0	7	2,191
2012	2,087	89	9	0	7	2,192
2013	2,091	89	9	0	7	2,196
2014	2,634	122	9	0	12	2,776
2015	2,129	93	9	66	9	2,306
2016	2,144	97	9	69	9	2,328

Note:
(1) Historical number of connections based on information provided by City Staff.

3.4.3 Historical Water Consumption

Historical annual water consumption data by customer class for the years 2011 through 2016 was obtained from the City's billing records and is presented in Table 3.4 as gallons per day (gpd). Unbilled metered consumption covers authorized water use for activities such as hydrant flushing, and is detailed further Section 3.4.3.1. During this period, overall water consumption has been generally consistent, with peak consumption in 2015. Residential water use dominates water consumption historically. General Residential accounts make up 94 percent of all customer accounts and 83 percent of total water consumption, as shown graphically in Figure 3.4 and Figure 3.5.

Table 3.4 Historical Water Consumption by Customer Class

Year	General Residential (gpd)	Commercial (gpd)	Industrial (gpd)	Manufactured Homes (gpd)	Public Lands (gpd)	Unbilled Metered (gpd)	Total (gpd)
2011	418,410	47,649	1,965	-	7,262	58,830	534,115
2012	442,222	52,418	2,933	-	6,321	67,827	571,721
2013	425,382	55,504	2,165	-	6,017	68,271	557,338
2014	452,844	52,554	1,783	-	6,113	61,603	574,897
2015	435,693	29,148	1,849	102,318	3,026	58,178	630,212
2016	361,671	57,790	1,882	62,453	6,426	52,867	543,089
Average	422,703	49,177	2,096	27,462	5,861	61,263	568,562
75th Percentile	440,589	54,766	2,115	46,840	6,399	66,271	574,103

Notes:
(1) Bimonthly water and wastewater consumption data by customer type was provided by City staff through 2015.
(2) Monthly water and wastewater consumption data by customer type was provided by City staff in 2016.
(3) Total water consumption by demand type was estimated using total water consumption and distributed relative based on wastewater consumption.

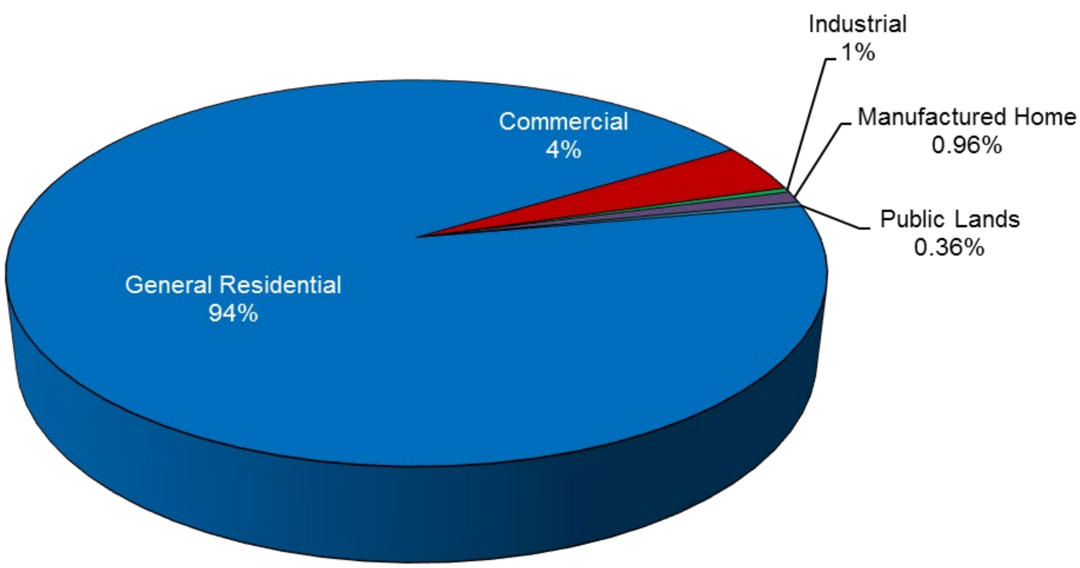


Figure 3.4 Average Percent of Accounts by Customer Class

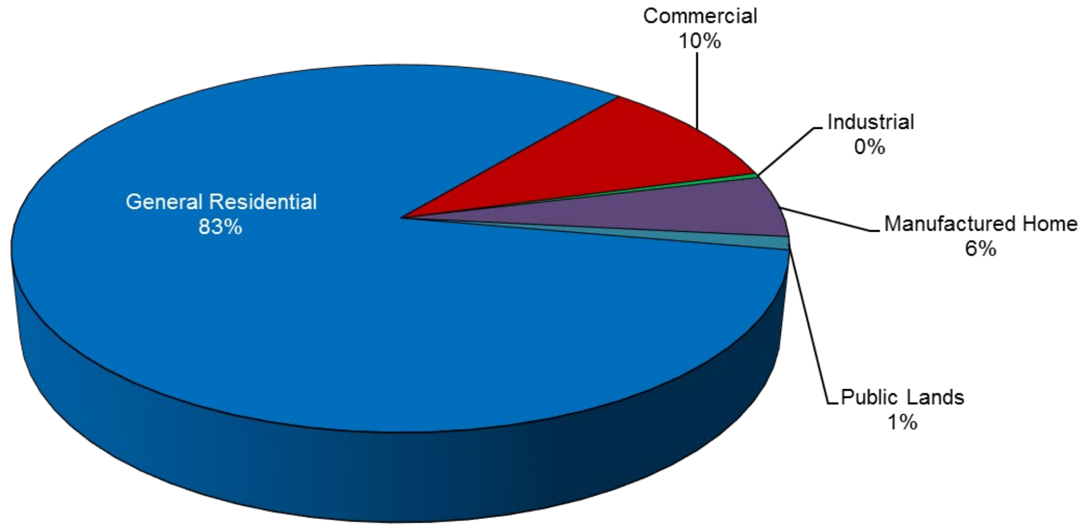


Figure 3.5 Average Percent Consumed by Customer Class

Water consumption on a per account basis has been relatively consistent for General Residential and Industrial water users, as presented in Table 3.5 and shown in Figure 3.6. Commercial and Public Land accounts showed a substantial decrease through 2015, before trending back up in 2016. Industrial, and to a lesser extent Commercial, water use are notably low compared to many utilities in the Northwest. Commercial water use is typically 800 to 2,000 gpd per account. Industrial is highly variable, but typically higher than commercial. Water use per account by Manufactured Homes has been greater than any other account type, but the trend in consumption is unclear after only two years as a designated account type.

Table 3.5 Historical Water Use per Account

Year	General Residential (gpd)	Commercial (gpd)	Industrial (gpd)	Manufactured Home (gpd)	Public Lands (gpd)	Unbilled Metered (gpd)	Total (gpd)
2011	200	542	218	--	1,037	58,830	244
2012	212	588	326	--	862	67,827	261
2013	203	625	241	--	860	68,271	254
2014	172	432	193	--	523	61,603	207
2015	205	312	214	1,546	343	58,178	273
2016	169	593	209	910	714	52,867	233
Average	194	515	234	1,228	723	61,263	245
75th Percentile	204	592	235	1,387	861	66,271	259

Note:
(1) Consumption per Account = Annual water demand (gpd) / No. of Connections.

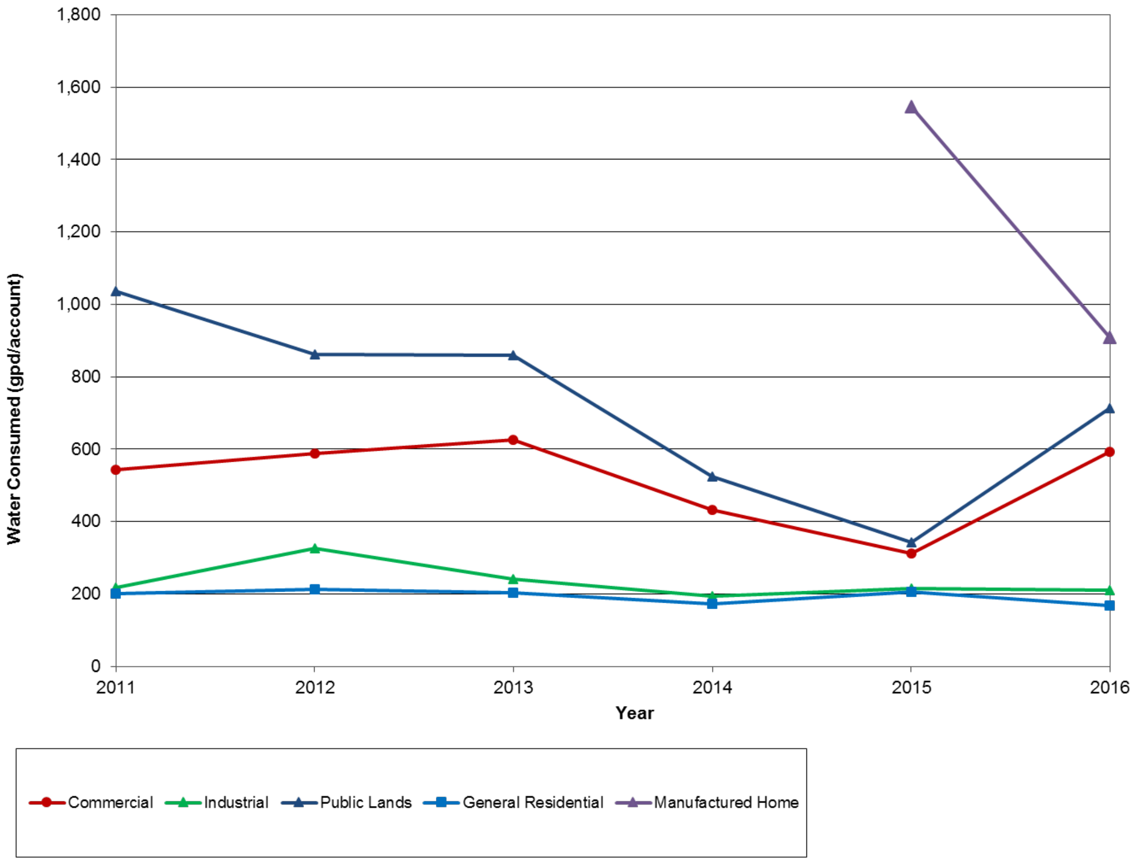


Figure 3.6 Historical Water Consumed per Account

3.4.3.1 Unbilled Metered Consumption

Unbilled metered consumption, as shown in Table 3.4, covers tracked and estimated water used by the City at the Keys Road and Miller Road treatment plants as well as water used for hydrant flushing. Water used at the two treatment plants encompasses backwash and filter-to-waste. The City also monitors private hydrant use though temporary meters, which are typically used for construction activities.

3.4.4 Historical Seasonal Water Consumption

Seasonal water consumption by customer class was evaluated based on billing records from 2016, which is shown in Figure 3.7. Water consumption peaks in the months of June, July, August, and September due to increased residential use and summer irrigation. General Residential water use drives peak summer demands on a volume basis.

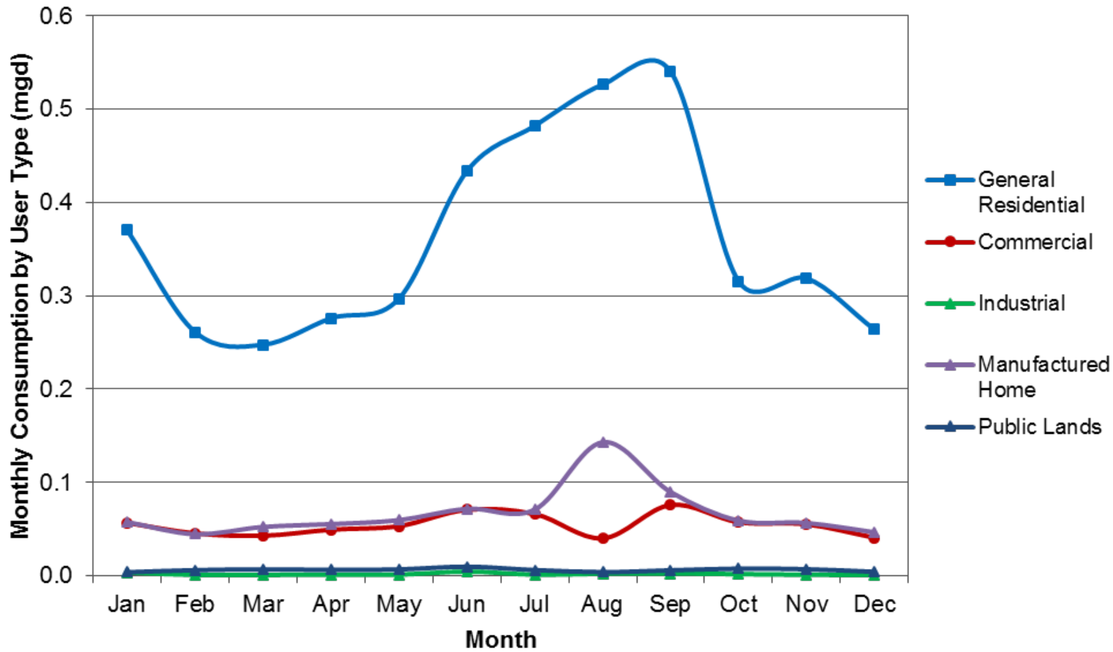


Figure 3.7 Seasonal Consumption by Customer Class (2016)

3.4.5 Water Losses

Water losses are estimated as the difference between total water supplied minus total authorized consumption. The American Water Works Association (AWWA) defines the following terms when examining water losses:

- Water supplied equals total metered source water production.
- Authorized consumption is the volume of water authorized for use by the water system. This includes both authorized metered consumption as well as other authorized unbilled consumption.
- Billed metered consumption is all metered consumption billed to customers as tracked through customer service meters.
- Unbilled authorized consumption is all authorized, unbilled consumption. Unbilled consumption includes unbilled metered and unbilled unmetered consumption. The City's unbilled authorized consumption includes metered consumption for activities such as plant backwash and unmetered consumption for hydrant flushing activities.

All water that is not authorized is considered water loss. Water loss includes both apparent and real losses. Apparent losses include water theft, meter inaccuracies, and data collection errors. Real losses are physical losses from the distribution system including reservoir overflows, water main breaks, and water main leaks.

Table 3.6 shows total water supplied, total authorized consumption, and water losses for the period of 2011 through 2016. Water loss has increased since 2012, with a maximum of 37.8 percent in 2015. This represents a significant loss of supply and potential revenue. For comparison purposes, water losses in Pacific Northwest municipal utilities are commonly below 15 percent. City staff are actively working to identify and resolve the issues.

The City has undertaken a number of efforts to identify apparent and real losses in the system. Acoustic water loss testing of the system was performed and known significant leaks within the City system have been fixed. In addition, nearly 80 percent of the meters in the system have been replaced with AMR meters or retrofitted with radios. City staff are further investigating meter calibration, accuracy, and maintenance of existing supply meters.

Table 3.6 Historical Water Losses

Year	Total Water Supplied(3) (mgd)	Billed Metered Consumption(2) (mgd)	Unbilled Authorized Consumption (mgd)	Total Authorized Consumption (mgd)	Water Losses (mgd)	Water Losses (percent)
2011	0.78	0.53	0.06	0.59	0.18	23.6
2012	0.78	0.57	0.07	0.64	0.14	17.8
2013	0.79	0.56	0.07	0.63	0.17	21.0
2014	0.87	0.57	0.06	0.64	0.23	26.8
2015	0.97	0.54	0.06	0.60	0.37	37.8
2016	0.85	0.48	0.05	0.54	0.31	36.9
Average	0.84	0.54	0.06	0.61	0.23	27.3

Notes:
(1) All production and consumption based on Data provided by City Staff.
(2) Metered consumption is based on metered customer use.
(3) Raw water supplied is based on metered water supplied to City's treatment plants.
(4) Unbilled authorized consumption includes City provided water use data for backwash, filter-to-waste, and hydrant flushing.

3.4.6 Equivalent Residential Units

ERUs are used to express water use by non-residential customers as an equivalent number of single-family residential customers. An ERU is the amount of water consumed by a typical full-time single-family residence.

Water use per ERU is calculated by dividing the total volume of water utilized in the residential customer class by the total number of active residential accounts. The volume of water used by other customer classes is divided by the residential water use to determine the number of ERUs utilized by other customer classes.

The average daily consumption per account for each customer class for years 2011 through 2016 was shown in Table 3.5. Residential consumption has been relatively steady over the time period, with a peak of 212 gpd in 2012. For planning purposes, the City selected an ERU value as the 75th percentile of the historic data, or 204 gallons per day per Equivalent Residential Unit (gpd/ERU). The 75th percentile was considered to be appropriate for sizing infrastructure improvements.

Historical Equivalent ERUs per account for each consumption class were calculated based on the selected ERU value, as shown in Table 3.7. As previously discussed, Industrial and to a lesser extent Commercial water use is notably low.

Table 3.7 Historical ERUs

Account Type	ERUs per Account
Residential	1.0
Commercial	2.9
Industrial	1.2
Manufactured Home	6.8
Public Lands	4.2

Note:
(1) ERUs shown are 75th percentile of historical data from 2011 through 2016.

3.5 Projected Water Demand

Projecting future water demand is one of the key elements of the water system planning process. Identification of system improvements, such as supply, pumping, storage, and piping requirements, are all related to demand projections. This section summarizes the ADD and MDD projections that were developed for the City's water system based on historical water demand trends and future growth assumptions. The demand projections are presented as a range in demands that may be experienced in the future. Low, medium, and high demand scenarios were developed by adjusting various demand projection parameters. The medium demand projection scenario is used to identify improvements described in Chapter 6. The low and high projection scenarios provide a sense of the extent of uncertainty in the demand forecasts.

3.5.1 Demand Projection Parameters

Future water use was projected using parameters developed from historical data and assumptions for future growth. Several parameters were used to project future ADD and MDD. The parameters, which are listed in Table 3.8, include ERU value, future customer water use, MDD to ADD peaking factor, and water loss percentage. For each demand projection parameter, low, medium, and high values were established corresponding to the respective demand scenario.

Demands for the potential annexation were projected using a different approach and are detailed in Section 3.5.2.

Table 3.8 Demand Projection Parameters

Parameter	Low	Medium	High
ERU Value (gpd/ERU)	204	204	204
Future Customer Water Use (percent of existing use)	95	95	100
MDD/ADD Peaking Factor	1.79	1.86	2.06
Annexation Peaking Factor	1.34	1.40	1.55
Water Loss (percent)	25.3	25.3	25.3

3.5.1.1 ERU Value

For all three demand projection scenarios, an ERU value of 204 gpd was used, which, as described previously, corresponds to the 75th percentile of historical data.

3.5.1.2 Future Customer Water Use

Future customer water use was based on equivalent ERU per account values, shown in Table 3.9. These values are based on the 75th percentile of historical data, except Industrial. Future Industrial water use was based on water use from existing industrial areas in the Northwest, as discussed in Section 3.5.2.

When projecting future ERUs for each customer type, a factor of 0.95 was applied. Future General Residential development was given an ERU value of 0.95, rather than 1.0, because it was assumed that new development will consume less water due to low-flow fixtures, water efficient appliances, smaller lot sizes, and other factors.

For the low demand scenario, it was assumed the City would pursue a conservation program to reduce water use. The selected ERU value of 204 gpd was reduced by one percent per year until it reached 175 gpd per day.

Table 3.9 Future Water Use by Customer Type

Account Type	Existing Accounts (ERUs per Account)	Future Accounts (ERUs per Account)
Residential	1.0	0.95
Commercial	2.9	2.8
Industrial	10.0	10.0
Manufactured Home	6.8	6.5
Public Lands	4.2	4.0

3.5.1.3 MDD to ADD Peaking Factor

The MDD to ADD peaking factor, used for the low demand scenario, of 1.79 is the 50th percentile of the historical peaking factors presented in Table 3.2. The City selected a peaking factor of 1.86 for the medium demand projection, equal to the 75th percentile of historical peaking factors. For the high demand projection, a peaking factor 2.06 was selected, equal to the 90th percentile of historical peaking factors, which is greater than all recent years except 2011.

3.5.1.4 Water Losses

The City's water losses have increased significantly in recent years. The 50th percentile water loss of 25.3 percent from historical data was used as a basis for all scenarios of the demand projections. The City is pursuing a water loss control program, with a goal of reducing losses to a target of 15 percent. The demand projections captured this intent and incorporated progress towards that goal in future projections, as presented in Table 3.10. Water losses were reduced by 1.0 percent annually for the low demand scenario until the loss target was achieved, and reduced by 0.5 percent annually for the medium and high demand scenarios. Based on these rates, it would take between 10 and 20 years to achieve the City's goal.

Table 3.10 Water Loss Goals

Parameter	Low	Medium	High
Average Water loss (percent)	25.3	25.3	25.3
Water Loss Target (percent)	15.0	15.0	15.0
Annual Water Loss Reduction (percent)	1.0	0.5	0.5
Years to Achieve Goal	10	20	20

3.5.2 Annexation Demands

The City has a proposed annexation area within its UGA, with future development located near the existing airport. City staff identified future land use designations and the expected acreage for new development. Demand projections for the annexation area followed a different approach from the areas with existing customers. Future development in the annexation area was broken into three different customer classes: annex commercial, annex industrial, and airport employment. City staff provided the total acreage for each customer class expected to develop in the ten year and twenty year planning timeframes.

For each development type, a water use rate, in ERUs per acre (ERU/acre), was used to project future water demands. For annex commercial, a water use rate of 2.9 ERU/acre was based on the City's existing commercial water demands. Annex industrial and airport employment were expected to be significantly different from the City's existing industrial accounts, so water use rates were based on an analysis of actual water use rates of other industrial and manufacturing areas in the Pacific Northwest. A water use rate of 5 ERU/acre was selected for annex industrial and airport employment; these values are used for all demand scenarios. A summary of the total acreage and projected water use for the annexation areas is shown in Table 3.11.

Table 3.11 Annexation Area Demand Summary

Development Designation	Annex Commercial	Annex Industrial	Airport Employment	Total
Expected Acreage Developed in 10 years	10	50	165	225
Additional Expected Acreage Developed in 20 years	10	100	166	276
Total Acreage Developed in Planning Horizon	20	150	331	501
Water Use Rate (ERU/acre)	2.9	5	5	
Water Use in 10-year Horizon (ERU)	37	386	1,051	1,475
Water Use in 20-year Horizon (ERU)	61	780	1,722	2,563

Development of all three annexation areas was projected to begin in 2017, with demands increasing linearly to the 10 and 20 year planning periods.

3.5.2.1 Annexation Peaking Factor

For future annexation areas, assigned MDD/ADD peaking factors were 75 percent of the factors established for existing areas of development. Future annexation areas are projected to be commercial and industrial land use, including increased airport employment. Irrigation requirements for the annexation developments are expected to be minimal, so a lower peaking factor was used.

3.5.3 Account Projections

Using the demographic growth rates, the number of existing accounts in each customer category was grown to project the future number of water accounts. General Residential, Manufactured Home, and Public Lands accounts were grown by the population growth rate. Commercial and Industrial accounts were grown by the employment growth rates established in the comprehensive plan. Growth of annexation area developments is as described previously. The projected number of future accounts was then used to develop ERU and demand projections, as described below.

3.5.4 ERU Projections

The ERU projections provide the basis for the ADD and MDD demand projections. The projected number of ERUs for the City was calculated by multiplying the projected number of accounts by the number of ERUs per account presented in Table 3.9. For all three demand projection scenarios, the total ERUs are projected to more than double by the end of the planning period, as shown in Table 3.12. These values include all new development in the potential annexation area.

Table 3.12 ERU Projections for Low, Medium, High Demand Scenarios

Year	2018	2023	2028	2033	2038
Low Demand (ERUs)	3,654	4,484	5,421	6,330	7,187
Medium Demand (ERUs)	3,654	4,484	5,421	6,330	7,187
High Demand (ERUs)	3,807	4,650	5,604	6,527	7,403

3.5.5 Projected Average and MDD

Average day demand projections for each customer class were calculated by converting consumption in ERUs to gpd using the ERU values presented in Table 3.12. Water losses were included, starting at 25.3 percent and decreasing to the target of 15 percent as shown in Table 3.9, to establish total ADD projections.

MDD projections were established by multiplying ADD projections by the appropriate ADD to MDD peaking factor for each demand projection scenario, as presented in Table 3.8. ADD and MDD projections for the low, medium, and high demand projection scenarios are presented in Table 3.13 and Table 3.14, respectively. ADD is projected to nearly double by 2038, with MDD projected to reach 2.9 mgd by 2038 in the medium scenario. Future demand projections for the entire planning period are presented in Figure 3.8.

Table 3.13 Projected ADDs for Low, Medium, High Demand Scenarios

Year	2018	2023	2028	2033	2038
Low Demand (mgd)	0.93	1.02	1.13	1.30	1.48
Medium Demand (mgd)	0.98	1.16	1.36	1.54	1.73
High Demand (mgd)	1.02	1.21	1.41	1.59	1.78

Table 3.14 Projected MDDs for Low, Medium, High Demand Scenarios

Year	2028	2023	2028	2033	2038
Low Demand (ERUs)	1.64	1.75	1.91	2.17	2.45
Medium Demand (ERUs)	1.79	2.08	2.40	2.67	2.97
High Demand (ERUs)	2.07	2.39	2.75	3.06	3.39

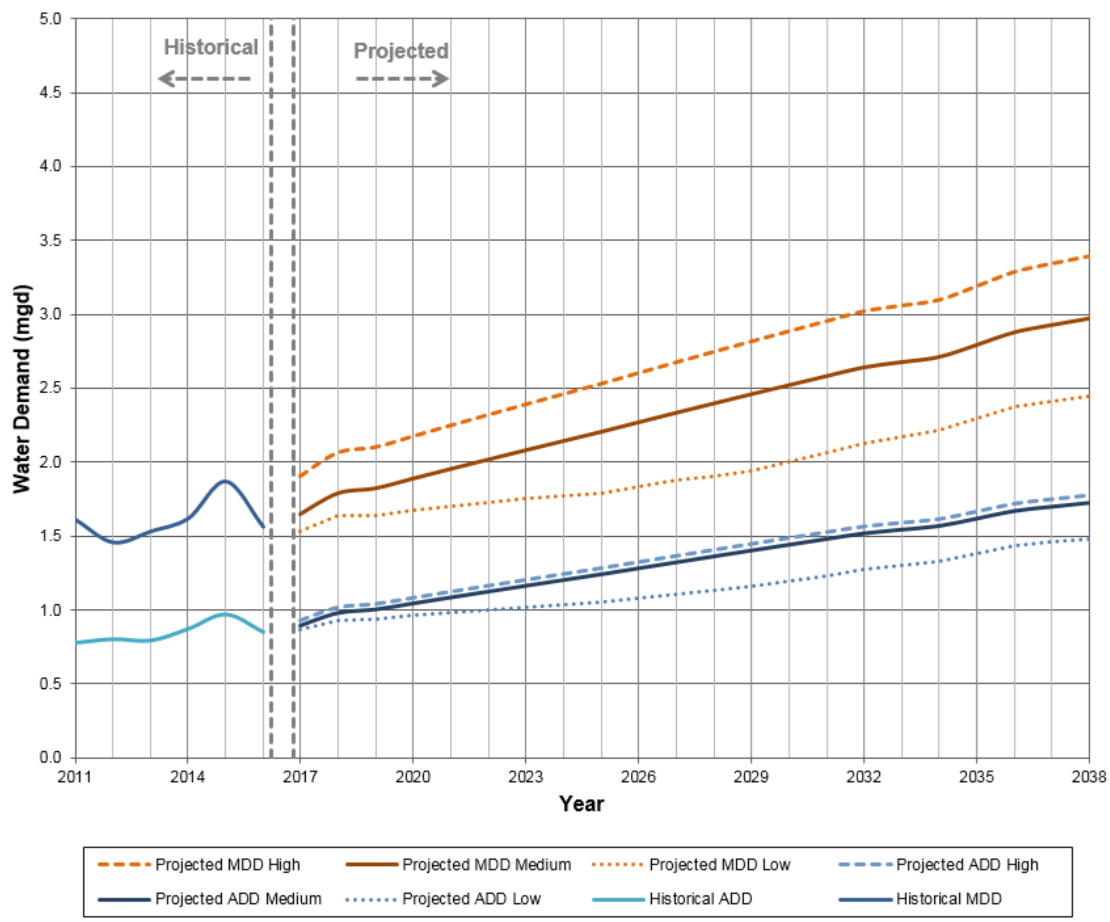


Figure 3.8 Historical and Projected Average and Maximum Day Demands

3.5.6 Projected Demand by Pressure Zone

A breakdown of historical demand and projected 2038 demand by pressure zone is shown in Figure 3.9 and Figure 3.10, respectively. Existing demands by pressure zone were estimated based on the anticipated growth in each pressure zone. Pressure Zone 1 serves the majority of the City's existing and projected customers. To predict future demand by pressure zone, future land use was used to attribute a percentage of the total growth of each customer class to each pressure zone. Demand distribution at the end of the planning period is projected to be similar to the existing distribution throughout the City's four pressure zones. Pressure Zone 1 dominates existing demands, and is the source of 90 percent of future growth in ADD as shown in

Table 3.15. This breakdown of future demands was used when performing a capacity evaluation of the system.

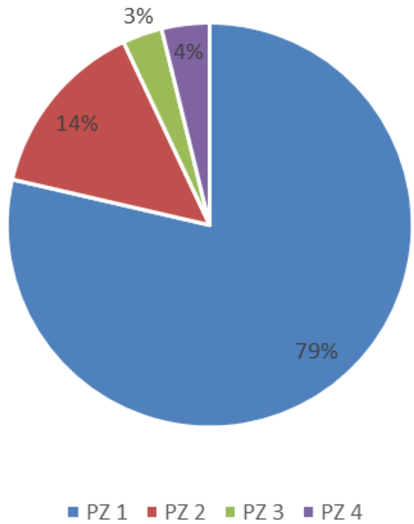


Figure 3.9 Existing Demand by Pressure Zone

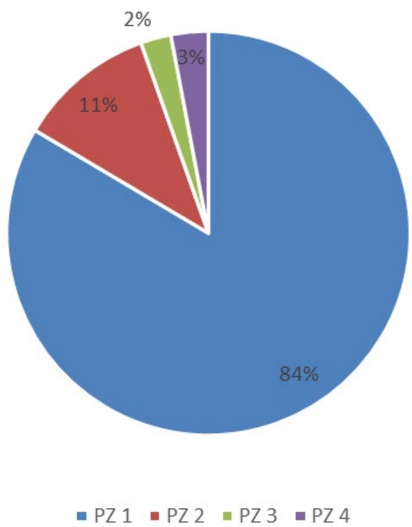


Figure 3.10 Future Demand by Pressure Zone

Table 3.15 Growth in ADD by Pressure Zone

Pressure Zone	Existing (mgd)	Future (mgd)	Growth (mgd)
1	0.67	1.42	0.73
2	0.12	0.19	0.06
3	0.03	0.04	0.01
4	0.03	0.05	0.02

Chapter 4
WATER RESOURCES

4.1 Introduction

This chapter provides an evaluation of the City of Scappoose's (City's) current and future water resources. The City's existing water supply sources are a combination of surface water supplies from diversion structures on South Fork Scappoose Creek, Lazy Creek, and Gourlay Creek and four groundwater wells (Dutch Canyon, Miller Road #1, Miller Road #2, and Miller Road #3). The City's existing water resources are compared against the demand projections developed in Chapter 3 to determine the City's long-term supply strategy. This chapter contains information from the 2018 Water Management and Conservation Plan (WMCP) that was developed concurrently with this master plan. Further details can be found in the WMCP as needed.

4.2 Water Rights Summary

The City's existing surface water and groundwater water rights are discussed below and are summarized in Table 4.1.

4.2.1 Surface Water

The City holds total water rights to withdraw a combined total of 9.0 million gallons per day (mgd) from all three surface water sources. Use of the surface water rights is seasonally limited by available streamflow during the summer and fall. All City rights are more senior than instream flow rights on the source streams, which are shown in Table 4.2.

4.2.2 Groundwater

The City's groundwater rights authorize a total withdrawal of 0.94 mgd within the South Scappoose basin and 1.94 mgd within the Jackson Creek Basin. The City has certificates for two of their five municipal water rights. The City has perfected Dutch Canyon Well No. 1's water right (G-8615). The City has applied for additional points of withdrawal to perfect its rights through construction of additional wells.

Table 4.1 Water Right Summary

POD Description	Approximate POD Location	Application Number	Permit Number	Beneficial Uses	Permitted Allocation (cfs)(3)	Allowed Rate under Development Limitations Condition and/or Perfected Rate of Certificate (cfs)	Permitted Allocation (mgd)	Permit Date	Certificate Number	Certificate Date	Certified Allocation (cfs)	Certified Allocation (mgd)	Completion of Development Date (Extended Completion Date)	Priority Date
Gourlay Creek (tributary to South Scappoose Creek)	NE ¼ SE ¼ , Section 12, T3N, R2W	S-8815	S-5813	Municipal Use	10.00	N/A	6.46	4/12/1923	5573	11/30/1925	10.00	6.46	N/A	1/24/1923
Lazy Creek (tributary to South Scappoose Creek)	POD 1: SE ¼ NW ¼ , Section 18, T3N, R2W	S-27859	S-25918	Municipal Use	1.50	N/A	0.97	3/16/1959	42700	12/5/1975	1.50	0.97	N/A	11/24/1958
South Fork Scappoose Creek (tributary to Scappoose Creek)	POD 2: NW ¼ SE ¼ , Section 7, T3N, R2W	S-27859	S-25918	Municipal Use	2.50	N/A	1.62	3/16/1959	42700	12/5/1975	2.50	1.62	N/A	11/24/1958
Dutch Canyon Area (1 well under each permit in South Scappoose Creek Basin)	NE ¼ SW ¼ , Section 13, T3N, R2W	G-9218 (Transfer T-12586)	G-8615	Municipal Use	0.89	N/A	0.58	8/31/1979	N/A	N/A	0.40	N/A	N/A	4/30/1979
	NE ¼ SW ¼ , Section 13, T3N, R2W Additional Points of Appropriation: NE ¼ SW ¼ , NE ¼ SW ¼, NE ¼ SW ¼, NE ¼ SW ¼, NE ¼ SW ¼ Of Section 13, T3N, R2W	G-15135 (Transfer T-12258)	G-17643 (supersedes G-15295)	Municipal Use	0.55	0.0(1)	0.36	12/20/2002	N/A	N/A	N/A	N/A	10/1/2050 (10/1/2050)	3/10/2000
Miller Road Area (3 wells in Jackson Creek basin)	POD 1 & 2: SE ¼ NW ¼ , Section 7, T3N, R1W Additional Points of Appropriation: SE ¼ NW ¼ , NE ¼ SW ¼, SW ¼ NW ¼, NW ¼ NW ¼, SE ¼ NW ¼ , Of Section 7, T3N, R1W	G-15792 (Transfer T-12284)	G-17644 (supersedes G-15491)	Municipal Use	2.23 Well #1	0.76(2)	1.44	9/15/2003	N/A	N/A	N/A	N/A	10/1/2007 (10/1/2050)	7/5/2002
				Municipal Use	0.67 Well #2	0.58(2)	0.43		N/A	N/A	N/A	N/A		
Oak Street Area (a well in Jackson Creek Basin)	NE ¼ SE ¼ , Section 12, T3N, R2W	GR-926 (claim)	GR-926 (claim)	Municipal Use	0.11	N/A	0.07	N/A	N/A	N/A	N/A	N/A	N/A	12/31/1950
Total					**18.46**	**1.34**	**11.93**				**14.00**	**9.05**		

Notes:
(1) As established by the "Development Limitations" condition in the Final Order issued December 12, 2014.
(2) As established by the "Development Limitations" condition in the Final Order issued August 29, 2014.
(3) cfs - cubic feet per second.

Table 4.2 Instream Flows Water Rights

Water Right Certificate Number	Water Body	Priority Date
59519	Gourlay Creek	1966
59688	South Scappoose Creek	1966

4.3 Ability to Supply

The Ability to Supply represents the City's supply capacity used in long-term planning. It is determined for normal conditions and with the surface water supply out-of-service, referred to as the redundant scenario. The Ability to Supply is compared to future demands to identify supply deficiencies and time improvements.

4.3.1 Existing Ability to Supply

The existing Ability to Supply represents the City's supply capacity for its existing surface water and groundwater supplies. It is the maximum production for a source considering the following:

- Water Right: Maximum water rights available.
- Diversion / Pumping Capacity: Physical diversion or pumping capacity for each individual surface water diversion or well, as provided by the City.
- Treatment Capacity: Total treatment capacity for the City's water treatment plants (WTPs).
- Operational/ Seasonal Limitations (mgd): Limitations on the use of supplies due to seasonal variations in available supply or operational requirements.

Surface water supplies and existing well fields are presented in Table 4.3. The City's total existing Ability to Supply is 1.49 mgd. Major source limitations include:

1. Surface water supply is limited to as low as 0.36 mgd in the summer and fall due to available stream flows.
2. The Miller Road and Dutch Canyon wells produce less than their water rights and available treatment.

Table 4.3 Existing Ability to Supply during Normal Conditions

Existing Supply	Permit #	Permitted Diversion		Diversion/ Production Capacity		Treatment		Operational/ Seasonal Limitations		Ability to Supply	
		mgd	cfs	mgd	cfs	mgd	cfs	mgd	cfs	mgd	cfs
Surface Water		9.05	14.0	2.00	3.09	1.01	1.56	0.36	0.557	0.36	0.557
Miller Road	G-17644	0.86	1.33	0.65	1.01	1.44	2.23	NA		0.65	1.01
Dutch Canyon	G-8615	0.58	0.897	0.48	0.74	1.15	1.78	NA		0.48	0.74
Total		**12.02**		**3.13**		**3.60**				**1.49**	

4.3.2 Reliable Ability to Supply

In addition to normal conditions, the City considers the redundant Ability to Supply, where the surface water supply is assumed to be out-of-service. The City conducts water supply planning based on the reliable ability to supply; acknowledging periodic maintenance and repair may be required at any time. The surface water supply is not considered a reliable supply during peak summer demands and assumed to be out-of-service for water supply planning. Surface water supplies are considered to be unreliable for supply planning due to:

- The diversion structures or transmission pipelines' remote locations and permitting requirements has historically resulted in extended supply outage when substantial maintenance and repair activities are required.
- While the City conducts preventative maintenance and maintains a spare parts stock, the Keys Road treatment plant has infrastructure that requires specialized contractors to repair; historically resulting in extended supply outage when required.
- The direct filtration treatment process is sufficient for typical conditions; however, unusual conditions, such as an algae bloom or a wild fire in its tributary area, would likely result in an extended supply outage.
- The Keys Road Treatment Plant is not seismically resilient. The City's groundwater sources are more resilient to seismic events.

Without the surface water sources, the City's existing reliable ability to supply is 1.13 mgd (1.75 cfs), as shown in Table 4.4.

Table 4.4 Existing Reliable Ability to Supply

Existing Supply	Permit #	Ability to Supply		Reliable Ability to Supply	
		mgd	cfs	mgd	cfs
Surface Water		0.36	0.557	0	0
Miller Road	G-17644	0.65	1.01	0.65	1.01
Dutch Canyon	G-8615	0.48	0.743	0.48	0.743
Total		**1.49**	**2.31**	**1.13**	**1.75**

4.4 Future Supply Needs

The City's future supply needs are calculated as the difference between the City's demand projections and its existing Ability to Supply. Future supply needs were evaluated for both normal conditions and under the City's redundancy scenario (surface water supply out-of-service). Ability to Supply was compared to the average day demand (ADD) and maximum day demand (MDD) from 2018 through 2038.

At this time, the City may not have sufficient supply to meet MDD supplies without the future Dutch Canyon Well No. 2 in both normal and reliability scenarios. As shown in Table 4.5, by 2038, the reliable supply deficiency during the MDD is anticipated to be approximately 1.48 mgd (2.29 cfs) in the normal scenario and 1.84 mgd (2.85 cfs) in the reliability scenario. Note, during the 2038 ADD, the reliable supply deficiency is anticipated to be approximately 0.6 mgd (0.928 cfs).

This deficiency will likely be met from reservoir storage, where reservoir levels can remain depressed for several days before a lower demand period allows storage to be replenished.

Completion of the new Dutch Canyon Well #2 will likely provide sufficient supply under normal conditions; however, the City will remain deficient under the redundancy scenario.

Table 4.5 Projected Future Supply Deficiencies with Existing Sources

Year	Projected MDD (mgd)	Existing Ability to Supply (mgd)	Existing Redundant Ability to Supply (mgd)	Projected MDD Supply Excess/Deficiency (mgd)	Projected MDD Redundant Supply Excess/Deficiency (mgd)
2018	1.79	1.49	1.13	-0.30	-0.66
2028	2.40	1.49	1.13	-0.91	-1.27
2038	2.97	1.49	1.13	-1.48	-1.84

To meet the demand projections, the City will need to more than double their Ability to Supply. This will require development of new sources of supply. The City's surface water supplies are not able to provide additional reliable supplies due to previously discussed limitations. The City's groundwater supplies can provide future supplies, given additional well capacity and green light water. The City's has requested green light water to fully exercise the full extent of the water rights using the currently permitted diversion locations. To reduce or delay new supplies, the City will continue to implement cost-effective conservation measures to reduce demands.

4.5 Water Loss Control

To aid in addressing the high water loss, the City will develop a specific Water Loss Program in the 2019/2020 fiscal year, including specific funding in its biannual budgets. This program focus is on reducing water loss in the City's system. It is complemented by mandatory conservation measures and existing enhanced conservation measures targeted City customers. The Water Loss Program has been formulated to achieve the City's 2-year Water Loss Control Plan, as well as meet 5-year benchmarks. The goal the program is to reduce water loss to 10 percent from the 2018 loss of 33 percent. The activities have been divided into those that address real and apparent losses. Real loss control activities aim to reduce the physical leakage from the system. Apparent loss control activities aim at reducing errors in water measurement and analysis, as well as unauthorized water use.

The City anticipates 13 percent of current water loss to be real losses, equating to approximately 0.13 mgd (0.20 cfs). This value is based on the water loss reduction that would been required to reduce water loss to 10 percent from a mix of real and apparent losses. Water loss control savings are included as a future "source" of supply in future supply planning. The progress of the program will be evaluated and additional activities added as needed annually as part of the City's planning and budgeting processes. The Water Loss Control Plan is in Appendix B.

Note, the cost of water loss control could not be estimated as part of this Plan, as the cause for the losses are not known.

4.6 Future Supply Options

The WMCP considered alternatives for the City to meet future supply needs. The City intends to make full use of their existing alluvial groundwater water rights at Miller Road and Dutch Canyon well fields and related treatment plant capacity. The city has permitted additional points of withdrawal at both sites to allow multiple additional wells to achieve full use of the water rights, if needed. Full use of the City's water rights and the Water Loss Control Plan provide sufficient supplies for all but a small amount of the water supply - 0.03 mgd (0.05 cfs). Since the City is not anticipated to be built out at that time, a new supply source will be needed to meet future growth beyond the 20 year period. The City has multiple options for potential new sources and plans to further investigate those options in the next 20 year. The City is not seeking water rights for these sources at this time.

To plan for the substantial costs associated with developing a new supply source, the City considered three potential City owned feasible sources of supply: groundwater, Multnomah Channel using a Ranney well, and, to a lesser extent, the existing surface water supply. Based on these sources, infrastructure improvements for each supply source were detailed, further considering the repair and replacement of aging infrastructure and expanding supply capacity. Additionally, an interconnection with St. Helens, who has available supply capacity, was considered as an alternative to developing additional City owned supply capacity.

The resulting supply options were:

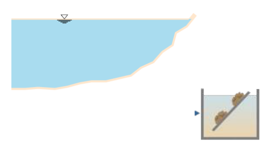
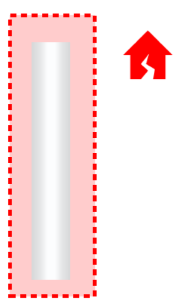
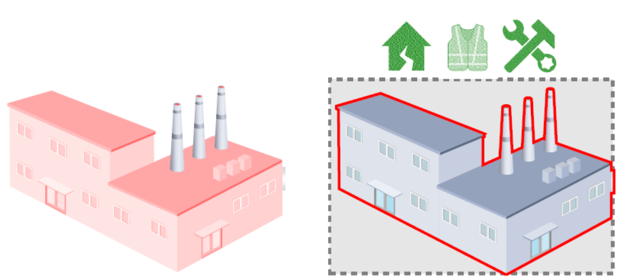
- Maintain surface water supplies through Repair and Replacement (R&R) of existing infrastructure.
- New Miller Road wells and expanded treatment capacity.
- New Dutch Canyon wells.
- New collector (Ranney) well on the Multnomah Channel and treatment.
- Interconnection with St. Helens (Ranney well on the Columbia River).

Details on each source are described in the following Table 4.6 through Table 4.10 and Figure 4.1 through Figure 4.6. The figures include a description, benefits, challenges, short-term improvement, long-term improvements and estimated capital costs. Implementation of sources was divided into two periods:

- Short-term: 2018 to 2028.
- Long-term: 2029 to 2038.

Cost assumptions and details on each source are provided in the following sections. The following sections also provide a discussion of the feasibility of pursuing each supply option.

Table 4.6 Existing Surface Water Supplies Supply Option Summary

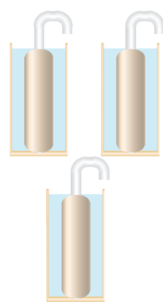
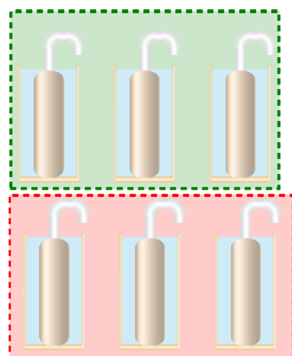
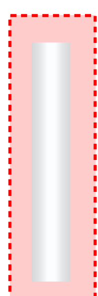
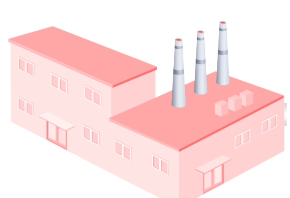
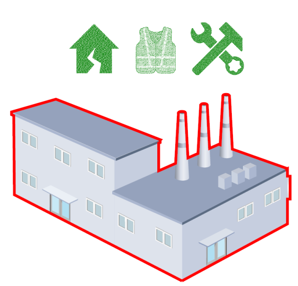
	Existing Surface Water Supplies	Description	Short-term Improvements	Long-term Improvements	Capital Costs Total: $12.6 million
Supply	 0.36 MGD	Maintain long-term use of City's existing surface water diversions from Lazy Creek, South Scappoose Creek, and Gourlay Creek. Available supply limited by streamflow. Use City's extensive surface water rights to extent possible.	Measure streamflow at each of the three surface water diversion structures. Continue regular maintenance for sediment and debris. Continue watershed protection efforts.	Operate supply based on streamflow measurements to maximize withdrawal of available supply, determined with the aid of a water rights attorney. Continue regular maintenance for sediment, debris, and other issues.	$25,000 for streamflow measurement.
Transmission	 2 MGD	Replace surface water transmission main from South Scappoose and Lazy Creek diversions (see Figure 4.1). Major capital construction project. Disruptive to community during construction. Seismic and resiliency concerns. Enhance ability to operate and maintain main.	Perform leak detection on existing raw water transmission mains. Install ports for cleaning sediment from the raw water mains.	Replace 25,000 LF(1) of 12.75 inch OD(1) steel raw water main from the South Scappoose Creek Diversion and along Dutch Canyon Road. Replace 4,200 LF of 8 inch CI(1) raw water main from the Lazy Creek Diversion to Dutch Canyon Road. Gourlay Creek: No anticipated R&R.	$170/LF construction cost for replacement of 8-inch CI(1) and 12.75-inch steel raw water main. $7.7 million total cost with contingency, planning/engineering, and admin.
Treatment	 1 MGD 1 MGD	Maintain Keys Road surface water treatment in the short-term. Replace with a new conventional surface water treatment facility on-site in the long-term. Gravity-fed supply has lower operating costs than pumped supplies. Advanced water treatment is not required based on existing water quality.	Make life-safety improvements. Repair and replace aging infrastructure. Implement seismic retrofits.	Design and construct a new 1.1 mgd conventional treatment plant at Keys Road. Decommission the existing Keys Road surface water WTP after new facility is on-line.	$1.1 million budget placeholder for short-term rehabilitation of Keys Road WTP. $2.25 per gallon for new conventional surface water treatment. Total cost of $3.8 million including contingency, planning/engineering, and admin.

Note:
(1) CI: Cast Iron LF: Linear Feet OD: Outer Diameter.

LEGEND: GREEN SHORT-TERM RED LONG-TERM SEISMIC IMPROVEMENTS LIFE-SAFETY IMPROVEMENTS REPAIR AND REPLACE IMPROVEMENTS

Figure 4.1 Transmission Main Replacements for Maintaining Surface Water

Table 4.7 New Miller Road Wells Supply Option Summary

	New Miller Road Wells	Description	Short-term Improvements	Long-term Improvements	Capital Costs Total: $6.9 million – $20.4 million
Supply	 0.65 MGD  2.1 MGD	Develop up to six new groundwater wells to be treated at the Miller Road WTP. Number of wells dependent on other supplies. Each well assumed to produce 0.36 mgd. Maximizes existing water rights. New well locations (see Figure 4.8). Water quality and yield at new locations. Additional water rights needed for long-term supply.	Acquire property adjacent to Cascade Tissue for potential futures wells, including proposed CZ-1 well. Drill test well for CZ-1, MP-1, and school wells in the next two years. Develop wells producing a total of 1.1 mgd by 2026.	Identify locations for up to 1.1 mgd of additional groundwater wells. Acquire water rights for future wells beyond a total capacity of 1.87 mgd (including existing wells). Develop 1.1 mgd of groundwater well supply.	$1.68 million per well including contingency, planning/engineering, and admin. Costs include land acquisition cost for 0.33 acres at $250,000/acre. No land acquisition costs included for MP-1 due to location in park.
Transmission	 2.1 MGD	Transmission mains from each well to Miller Road WTP. Length dependent on well locations. Up to 10,000 feet transmission main could be required from school well locations.	1,500 LF of 8 inch transmission main for Miller Road for first three wells to be developed.	Long-term wells assumed to be developed from School Wellfield locations. 10,000 LF of 12 inch transmission main for first drilled well at School wellfield. 1,500 LF of 8 inch transmission main for additional wells developed at the School wellfield.	$180/LF construction cost for 8-inch transmission main. Total cost of $418,500 for each 1,500 LF, 8-inch transmission main including contingency, planning/engineering, and admin. $220/LF construction cost for 12-inch transmission main. Total cost of $3.4 million for 12-inch main from School wellfield.
Treatment	 1.8 MGD  1.7 MGD	Expansion of existing Miller Road WTP to add up to two new filters after existing treatment capacity exhausted (see Figure 4.2). Existing treatment capacity available for additional wells, if water quality comparable to existing wells. Filter expansion requirements dependent on well yield and water quality.	Make life-safety improvements. Repair and replace aging infrastructure. Implement seismic retrofits.	Design and construct an expansion of up to 1.8 mgd for the existing Miller Road WTP. New backwash basin (100,000 gallons).	$0.8 million budget placeholder for short-term rehabilitation of Miller Road WTP. Unit construction cost for green sand filter treatment expansion of $1.5/gallon. $1/gallon unit construction cost for backwash basin. Total cost of $4.2 million for 1.8 mgd treatment plant expansion and backwash basin including contingency, planning/engineering, and admin.

LEGEND: GREEN SHORT-TERM RED LONG-TERM SEISMIC IMPROVEMENTS LIFE-SAFETY IMPROVEMENTS REPAIR AND REPLACE IMPROVEMENTS

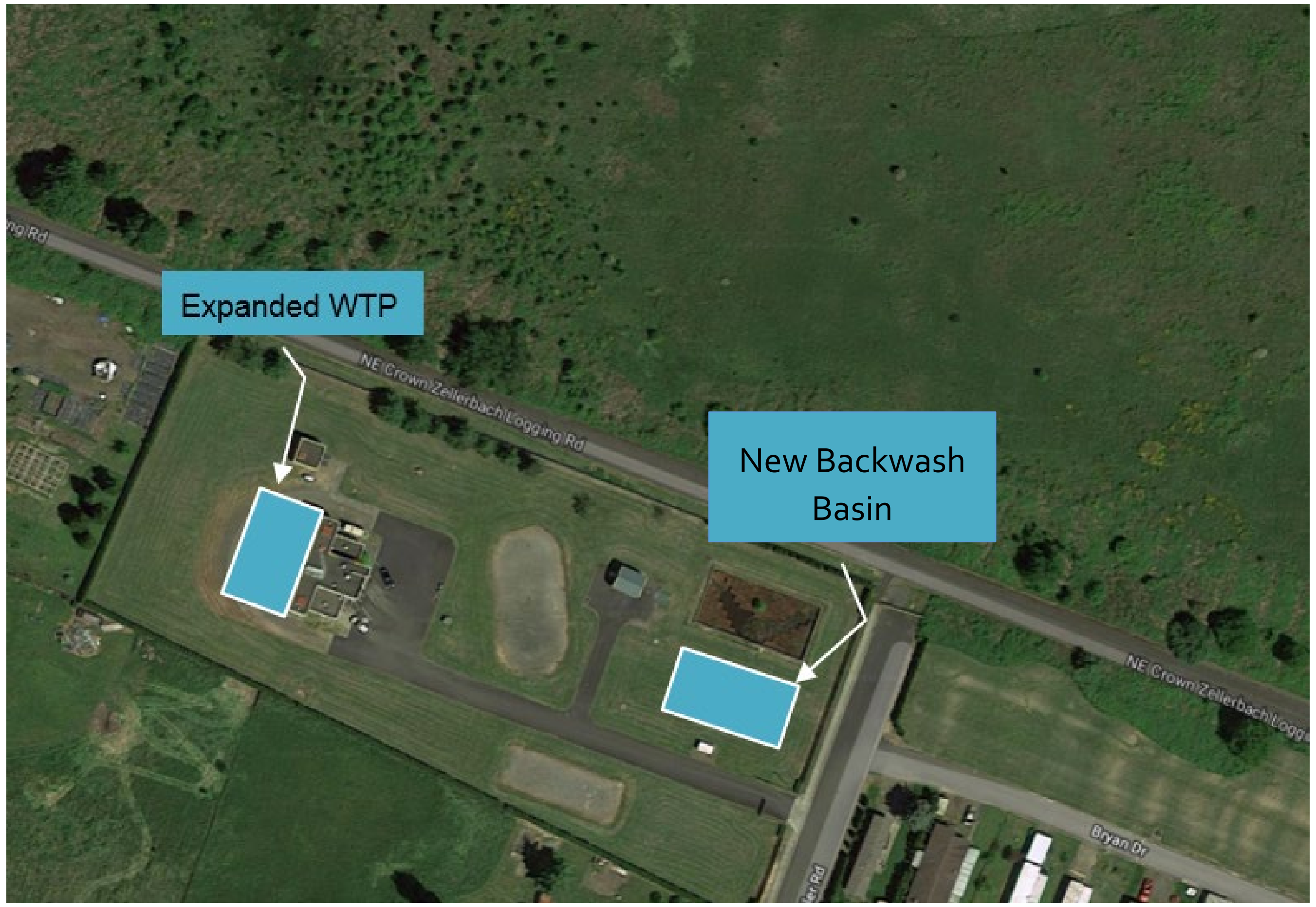
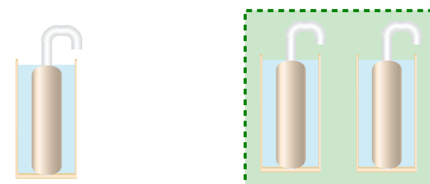
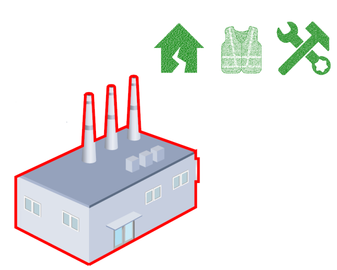


Figure 4.2 Expanded Miller Road Groundwater Treatment

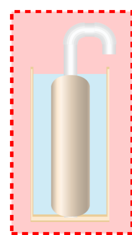
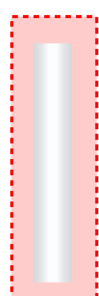
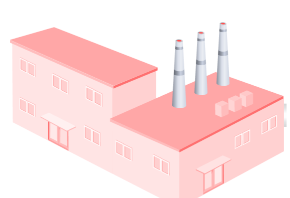
Table 4.8 New Dutch Canyon Well Supply Option Summary

	New Dutch Canyon Well	Description	Short-term Improvements	Long-term Improvements	Capital Costs Total: $1.2 million – $3.3 million
Supply	 0.48 MGD 0.65 MGD	Depending on final yield of Dutch Canyon Well #2, develop a third groundwater well near the existing Dutch Canyon wells. Maximizes existing water rights. New site for a third well should avoid interference with existing wells. Limited available property in area surrounding Dutch Canyon wells (see Figure 4.3).	Complete Dutch Canyon Well #2 development. If total well yield from existing Dutch Canyon wells does not maximize water rights, develop a third well. Acquire property near existing wells for third well.	None	$0.5 million for Dutch Canyon Well #2 mechanical, electrical, instrumentation, and controls. Total cost of $1.68 million for a third Dutch Canyon well including contingency, planning/engineering, and admin. Costs for third Dutch Canyon well include land acquisition cost for 0.33 acres at $250,000/acre.
Transmission	 1.1 MGD	Construct transmission main to connect new Well 3 site to existing raw water piping to Keys Road groundwater WTP.	8-inch transmission main from new well site, assumed to be 1,500 LF.	None	$180/LF construction cost for 8-inch transmission main. Total cost of $418,500 including contingency, planning/engineering, and admin.
Treatment	 1.1 MGD	Rehab and maintain use of existing Keys Road groundwater WTP.	Rehab and cover existing filter. Make life-safety improvements. Repair and replace aging infrastructure. Implement seismic retrofits.	None	$0.7 million budget placeholder for short-term rehabilitation of Keys Road groundwater WTP.

LEGEND: GREEN SHORT-TERM RED LONG-TERM SEISMIC IMPROVEMENTS LIFE-SAFETY IMPROVEMENTS REPAIR AND REPLACE IMPROVEMENTS

Figure 4.3 Dutch Canyon Wells and Surrounding Area

Table 4.9 New Ranney Collector Well Supply Option Summary

	New Ranney Collector Well	Description	Short-term Improvements	Long-term Improvements	Capital Costs Total: $12.4 million
Supply	 1 MGD	Develop a new Ranney collector well adjacent to the Multnomah Channel near Chapman Landing (see Figure 4.4). Greater supply capacity than City's groundwater wells (~3 mgd). Water quality unknown. Pumping capacity of 1 mgd, expandable to 3 mgd.	Conduct hydrogeological investigate to determine collector well sizing and yield. Water quality sampling of test well water and Multnomah Channel.	Construct Ranney well and pump house to provide 1.1 mgd of supply, expandable to 3 mgd.	$4.3 million for Ranney collector well and 1.1 mgd of pumping capacity including contingency, planning/engineering, and admin. Costs include land acquisition cost for 1.0 acre at $250,000/acre.
Transmission	 3.4 MGD	Transmission main to new surface water treatment facility located at Miller Road. Transmission main to Miller Road WTP location on bike path along NE Crown Zellerbach Logging Road (See Figure 4.4).	None	7,300 LF of 16-inch main along bike path from Chapman Landing to Miller Road.	$240/LF construction cost for 16-inch transmission main. Total cost of $2.7 million for raw water transmission main along bike path to Miller Road including contingency, planning/engineering, and admin.
Treatment	 1.1 MGD	New advanced surface water treatment facility. Advanced treatment may be needed depending on water quality. (See Figure 4.5).	None	Construct new surface water treatment facility with Ranney well. Treatment includes conventional sand filtration and advanced treatment, ozone and granular activated carbon (GAC).	$3.25 per gallon construction cost for advanced surface water treatment. Total cost of $5.4 million for new 1.1 mgd advanced surface water treatment facility at Miller Road including contingency, planning/engineering, and admin.

LEGEND: GREEN SHORT-TERM RED LONG-TERM SEISMIC IMPROVEMENTS LIFE-SAFETY IMPROVEMENTS REPAIR AND REPLACE IMPROVEMENTS

Figure 4.4 Ranney Collector Well Location and Transmission

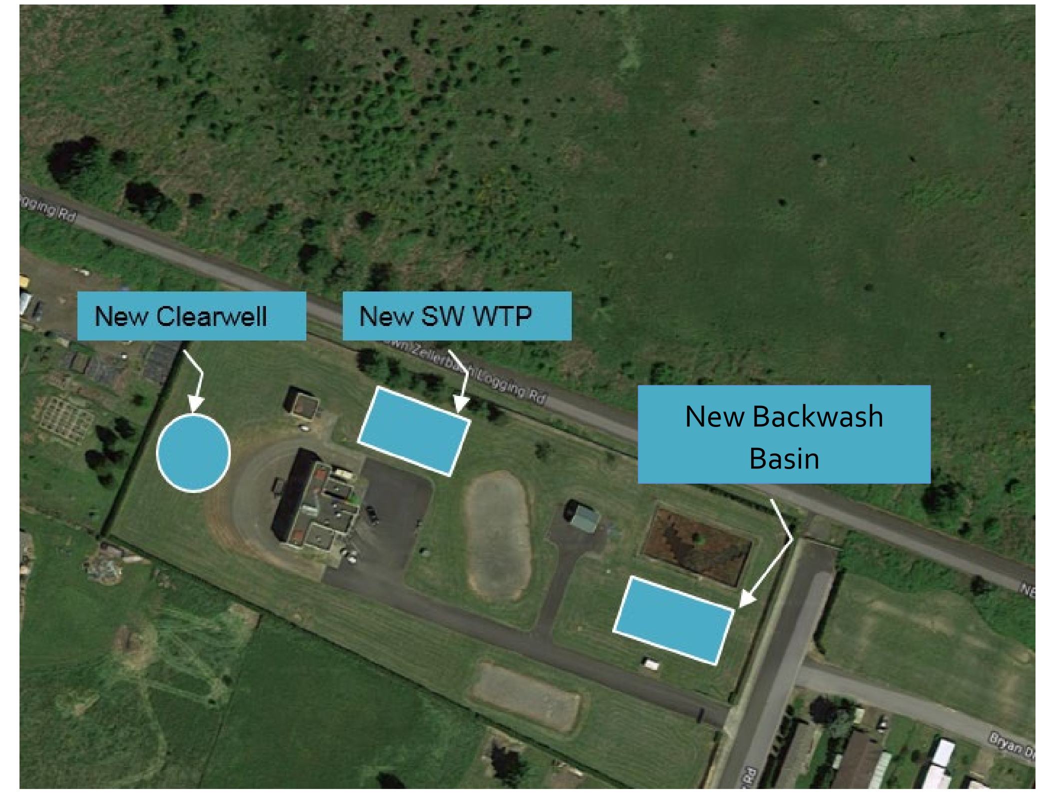
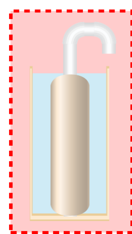
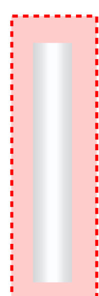
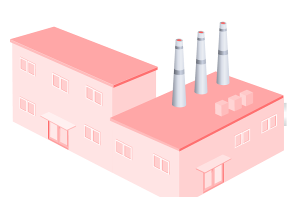


Figure 4.5 New Miller Road Surface Water Treatment Plant, Clearwell, and Backwash Basin

Table 4.10 Interconnection with St. Helens Supply Option Summary

	Interconnection with St. Helens	Description	Short-term Improvements	Long-term Improvements	Capital Costs Total: $33.6 million
Supply	 1 MGD	Interconnection with St. Helens to provide an initial 1 mgd of supply, up to 3 mgd in the future. Utilize existing St. Helens water supply. Known water quality and yield.	Determine buy-in and other costs associated with the existing St. Johns Ranney Well.	Supply expansion, if necessary.	Supply costs passed on through capital buy-in costs and/or volumetric rates. Assumed $3.9 million, including contingency, planning/engineering, and admin, for supply cost buy-in (See Alternative 3).
Transmission	 3.4 MGD	Transmission main to connect City of Scappoose system with St. Helens distribution system funded and constructed by NW Natural. Lengthy transmission main to connect to St. Helens system (5.5 – 10 miles). Supply redundancy concerns with single transmission main.	Identify project costs and right-of-way.	5.5 to 10 miles of 18-inch – 24-inch transmission main from St. Helens to existing distribution system. Construct booster pump station (BPS) with two redundant pumps for 1 mgd supply and one jockey pump for low demand periods. Expand pump station with two additional pumps to reach 3 mgd of capacity.	$310/LF construction cost for 24-inch transmission main. Total estimated cost of $21.6 million for 8.5 miles of 24-inch transmission main, including contingency, planning/engineering, and admin ($310/LF). Costs for NW Natural funding and project delivery method unknown. $1.1 million for BPS and pumps for the short-term. $1.6 million for long-term BPS expansion.
Treatment	 1.1 MGD	Buy-in for treatment capacity at St. Helens surface water treatment facility. Booster station to supply water from St. Helens.	Determine buy-in and other costs associated with water treatment.	Treatment expansion, if necessary.	Anticipated capital buy-in costs of $2 per gallon to $4 per gallon. $5.4 million for new surface water treatment facility, including contingency, planning/engineering, and admin, to account for treatment cost buy-in (See Alternative 3). Volumetric charges unknown.

LEGEND: GREEN SHORT-TERM RED LONG-TERM SEISMIC IMPROVEMENTS LIFE-SAFETY IMPROVEMENTS REPAIR AND REPLACE IMPROVEMENTS

Figure 4.6 Assumed Transmission Main Path from St. Helens to City of Scappoose

4.6.1 Cost Estimating Assumptions

Planning-level cost estimates were developed for supply, transmission, and treatment components for each supply option. Costs provided are planning level estimates only and should be refined during pre-design of the projects. The cost estimates developed in this chapter are American Academy of Cost Engineers (AACE) Class 4 estimates. Class 4 estimates are budget level estimates. Actual costs may vary from these estimates by -30 percent to +50 percent.

All costs are in February 2018 dollars. No inflation rate is applied to the cost of these supply options. This allows project costs to be inflated as warranted in the future. The Engineering News-Record (ENR) U.S. 20-City Construction Cost Index for February 2018 is 10,889.

The cost estimates were based on construction costs inflated using cost factors shown in Table 4.11.

Table 4.11 Cost Factors

Cost Factor	Description	Factor
Contingency	Costs that may occur due to uncertainty in project scope and conditions.	30%
Planning/Engineering and City Admin	Cost for planning and design of project as well as City administration costs for completing the project.	25%

Cost estimates for project components are included as part of the CIP presented in Chapter 8. Electronic versions of the CIP tool were provided to the City to update project cost estimates as needed in the future.

4.6.1.1 Supply

Table 4.12 provides cost estimates for new groundwater or Ranney collector well supplies. Cost estimates for new supplies included costs for land acquisition.

Estimated costs for new groundwater wells include a production well, site work, a structure, all mechanical and electrical equipment, and a back-up generator. Water right acquisition and test wells are not included in these costs.

Ranney collector well development costs include the following:

- $1,400,000 for construction of the collector well.
- $1,000,000 for pump house to pump water from the collector well to the treatment plant.
- $100,000 for a back-up generator.

Land acquisition was estimated at $250,000 per acre of land. New groundwater wells were assumed to require 1/3 acres and the Ranney collector well was assumed to require 1 acre.

Table 4.12 Supply Costs

Supply	Construction Cost
Groundwater well	$1,000,000
Ranney collector well	$2,500,000

4.6.1.2 Transmission

Transmission main unit construction costs are presented in Table 4.13. These unit costs assume open-trench construction and include pavement cutting, excavation, hauling, shoring, pipe materials and installation, backfill material and installation, and pavement replacement.

Acquisition, easements, and right-of-way (ROW) may be required for some of the recommended projects. For the purpose of these cost estimates, pipeline corridors were assumed to be in public ROW, and do not require land acquisition.

Costs for replacing existing surface water transmission main were developed based on initial discussion with the City. A single unit cost was assumed for this work, with no differentiation made for pipe size, as installation is the major driver in costs. Costs for polyvinyl chloride (PVC) slip lining were based on engineering judgment.

Table 4.13 Transmission Costs

Element	Unit	Unit Construction Cost
8 inch Pipe	LF	$180
10 inch Pipe	LF	$200
12 inch Pipe	LF	$220
16 inch Pipe	LF	$240
18 inch Pipe	LF	$260
24 inch Pipe	LF	$310
8 inch Fusible PVC Slip Line	LF	$100
Dutch Canyon Raw Water Main	LF	$170

4.6.1.3 Treatment

Planning level budget estimates for rehab and improvements for the City's existing treatment facilities are as follows:

- $1.1 million for short-term rehabilitation of Keys Road surface water WTP.
- $0.7 million for short-term rehabilitation of Keys Road groundwater WTP.
- $0.8 million for short-term rehabilitation of Miller Road WTP.

Note, see Chapter 5 and Chapter 8 for up-to-date budget estimates and details on improvements. Costs for new surface and groundwater treatment are summarized in Table 4.14.

Costs for groundwater treatment expansion at Miller Road WTP include costs for new filters, pumps, expansions of chemical systems, electrical, instrumentation and controls, and extending the existing building footprint. Expansion of Miller Road WTP also includes construction of an additional backwash basin. Costs for the backwash basin include site work, a belowground basin, and piping to connect the backwash basin to the existing facility.

Surface water treatment costs cover the cost for construction of a new conventional sand filtration plant. Costs include site work, a structure, filters, chemical systems, mechanical, electrical, and instrumentation and controls. Advanced surface water treatment may be needed to address a range of water quality issues that may be present in a new source and includes the costs for a conventional sand filtration plant with the addition of ozone and granular activated carbon.

Table 4.14 Treatment Costs

Supply	Unit	Unit Construction Cost
Groundwater Treatment Expansion	Gallon	$1.50
Surface Water Treatment	Gallon	$2.25
Advanced Surface Water Treatment	Gallon	$3.25
Ground Storage	Gallon	$1.00

4.6.1.4 Booster Pump Stations

BPS costs were included as part of the Ranney collector well and St. Helens interconnection supply options. BPS construction costs were estimated using a unit construction cost based on the number of pumps and horsepower (HP) of the pumps. Table 4.15 provides the unit construction costs used. Unit construction costs include site work, pumps, a structure, all mechanical and electrical equipment, and a back-up generator.

Table 4.15 Pump Station Costs

Horsepower	Unit	Unit Construction Cost
0 to 199 HP	Per HP per Pump	$4,100
200 to 349 HP	Per HP per Pump	$3,300
350 to 649 HP	Per HP per Pump	$2,500
>650 HP	Per HP per Pump	$1,700

4.6.1.5 St. Helens Interconnection

Cost estimates for transmission and pumping for the St. Helens interconnection were developed based on preliminary investigation. Transmission main head loss will require the construction of a BPS. The transmission main was assumed to be 8.5 miles of 24 inch pipe.

Initially, the BPS was assumed to consist of one 30 HP pump for low demand supply and two, 75 HP pumps to provide up to 1 mgd of supply. Future expansion to supply 3 mgd was assumed to require two additional 125 HP pumps. Pump station sizing assumed no head was provided from St. Helens treatment plant, and will need to be revised based on actual plant operation.

It is anticipated the City will have to pay a capital buy-in cost for St. Helens' existing supply and treatment infrastructure. For the purpose of this cost estimate, these costs were assumed to be equal to the cost of developing a Ranney collector well, $3.9 million, and associated 1.1 mgd treatment facility, $5.4 million.

4.6.2 Considerations for Future Supply Options

To better understand the future supply options, it is recommended the City perform investigative activities to better understand the available supply capacity and water quality. Key considerations to determine the feasibility of supplies and refine cost estimates are presented in this section.

4.6.2.1 Extent of Available Surface Water

Expansion of the City's existing surface water supplies is likely to be limited by available streamflow during the summer and fall. It is recommended the City begin monitoring stream flows at each surface water diversions to better understand available supply. The City should consult with a water rights attorney to determine available supplies based on the stream flow measurements.

4.6.2.2 Additional Groundwater Wells

Expanding production from Miller Road and Dutch Canyon sources requires development of new groundwater wells. Historically, the City has struggled to develop wells with both acceptable yield and water quality. The City and their hydrogeologists, GSI Water Solutions, Inc., recently identified potential well locations, which are presented in Figure 4.7. Future wells were sited considering the following:

- Oregon law requires new groundwater wells be at least 1/4 mile from surface water bodies (streams, lakes, etc.). This setback severely limits available areas for new wells.
- The City goal for well yields are least 0.36 mgd (250 gallons per minute [gpm]). Areas east of Highway 30 are believed to provide higher yields due to favorable hydrogeology.
- Consideration of future land uses, including the Airport's runway exclusion zone adjacent to the Miller Road treatment plant.

Note, Wells CZ-1, CZ-2, CZ-3, and MP-1 are permitted alternative points of withdrawal for the Miller Road wells. HS-1. PE-1, and PE-2 are not associated with an existing water right.

Historical test wells that were found to have unacceptable yield or water quality are also shown on the figure. Yield and water quality varies considerably, as presented in Table 4.16. Drilling of test wells are recommended in the short-term to confirm future well sites. Based on the test well findings, the City should update the costs for future groundwater supplies.

Table 4.16 Well Yield and Water Quality of Test Wells

Well Number	Location	Year Drilled	Well Yield	Iron Concentration (ppm)(1)	Manganese Concentration (ppm)(1)
1	Dutch Canyon	1978 (Active)	330 gpm	0.03 – 3.4 Average:0.43	Unknown
2	52212 SW Keys Rd (Keys Road WTP)	2000 (Abandoned)	0.3 gpm	Unknown	Unknown
3	SE 8th Court off of Elm Street	1997 (Abandoned)	470	173	4.8
4	Creek View Place (Park)	1996 (Abandoned)	900	8	Unknown
5	34066 Bernet Drive	1997 (Abandoned)	700	486	28.8
6	Miller Road	2001 (Active)	220	0.08 – 1.8 Average: 1.2	Unknown

Note:
(1) ppm: parts per million.

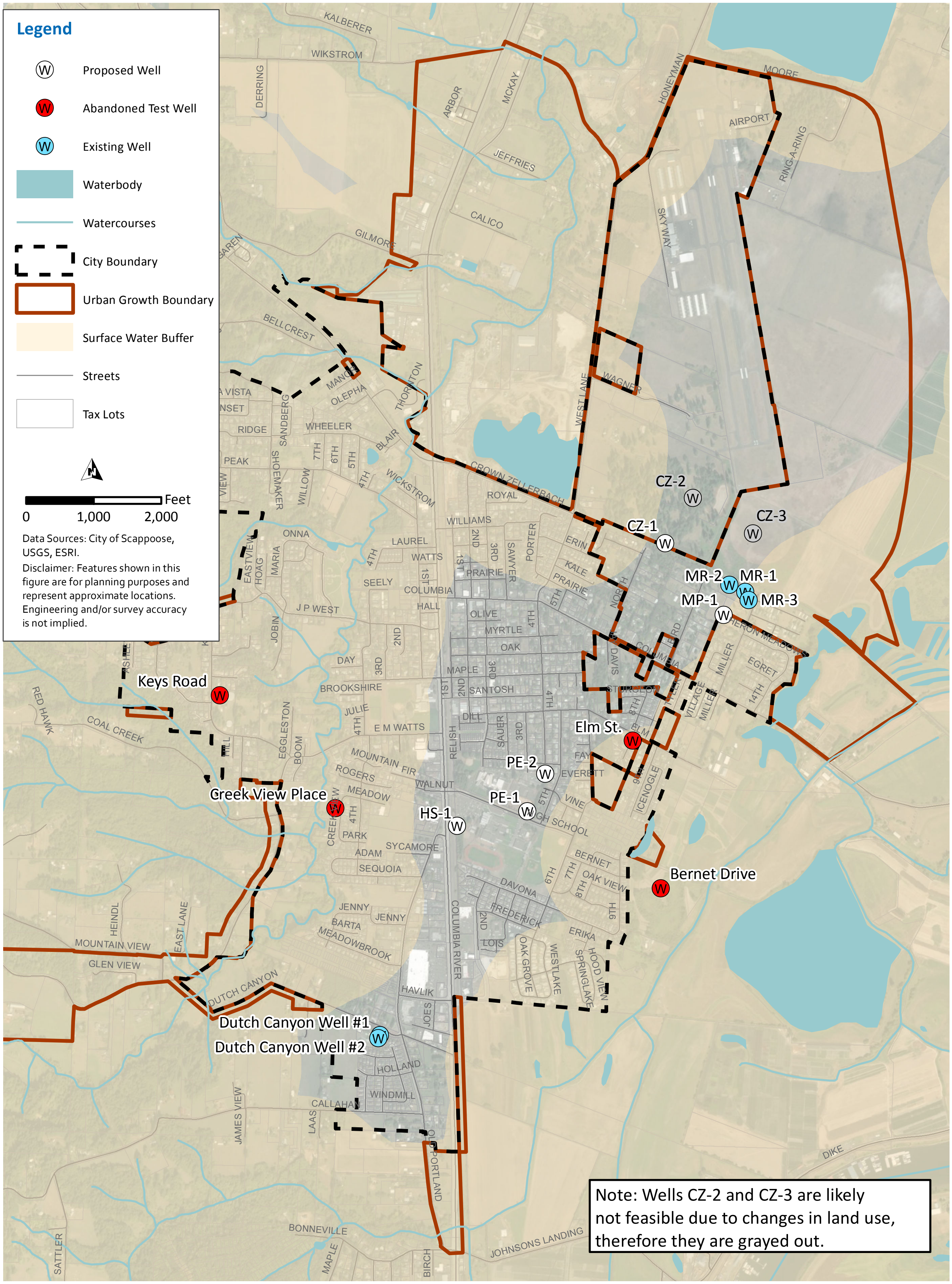


Figure 4.7 Existing and Potential Well Locations

Miller Road Wells

The map also includes previously identified sites for future wells and abandoned test wells, where viable sites are highlighted:

- The City submitted a greenwater right application for an additional Miller Road well (CZ-1, CZ-2, CZ-3, and MP-1). Due to the Airport's runway exclusion zone and the Cascade Tissue property, only MP-1 and CZ-1 are considered developable at this time. It is recommended the City consider if wells may be developed in additional adjacent areas.
- City should verify with the Port of St. Helens the ability to maintain wells near the airport runway and approach zone. Cranes and well drilling equipment will be required during construction and for maintenance activities.
- The potential for wells on Scappoose School District properties near the High School. Approximately 10,000 feet of transmission would be required to convey raw water supplies to Miller Road WTP. Alternatively, onsite treatment may be cost effective depending on water quality, given sufficient available land.
- Additionally, they may be required to address declining yield in the existing Miller Road wells. The City cleaned the wells and conducted maintenance on the well pumps, but observed limited changes in yield. Approximately 0.22 mgd (0.34 cfs) can be recovered from well rehabilitation.

Wellhead protection requirements for new wells will impact future development; City planning should be considered during well siting, especially near Miller Road WTP.

Dutch Canyon

The City is in the process of finalizing development of a second Dutch Canyon well. The current assumption is the existing well and new well will provide a combined 500 gpm of supply. The City's water rights allow for up to 650 gpm of production. A third well, in the vicinity of the existing Dutch Canyon wells, could be developed to maximize existing water rights and was included as a future supply option. It is unlikely a third well can be located on the Dutch Canyon well site without interfering with the existing wells. It is recommended the City purchase a nearby parcel in the short-term, as the surrounding area is rapidly developing. The City should finalize the final combined yield of the two existing wells. Should the well yield maximize the City's existing water rights, development of an additional well would not be needed.

Note, if favorable yield and water quality, Dutch Canyon Wells can be developed instead of Miller Road. An onsite high-rate iron and manganese system should be considered for future expansion. Sewer capacity for backwash is a major consideration.

The City has observed a decline in yield from Dutch Canyon Well No. 1 of approximately 0.1 mgd (0.155 cfs). The City is working with a hydrogeologist to determine how to rehabilitate Dutch Canyon Well No. 1 to regain the lost yield.

4.6.2.3 Feasibility of Ranney Collector Well

The City could potentially locate a Ranney Well at Chapman Landing, which is located east of the Miller Road WTP on the Multnomah Channel. The feasibility of this supply needs to be confirmed through hydrogeological and water quality investigations. Depending on the hydrogeology, the Ranney well may collect groundwater or groundwater under the influence of surface water (the nearby Multnomah Channel). Full surface water treatment is regulatorily required for groundwater under the influence of surface water. It is anticipated that advanced treatment (granular activated carbon and ozone) would be required.

It is recommended to conduct a hydrogeological test at Chapman Landing to evaluate the potential capacity and sizing of a Ranney collector well. Collect water quality information from the test well and Multnomah Channel.

Note, depending on the hydrogeology, a wellfield of traditional vertical wells may be used in place of a Ranney well. If feasible, it allows the City to better add capacity in time with development.

The area surrounding Chapman Landing is located within a floodplain. The levee along the Multnomah Channel is not certified and the level of flooding protection is unknown. Due to the flooding risk, it is recommended that any new treatment facility be located outside the floodplain.

4.6.2.4 Investigation of St. Helens Interconnection

The City of St. Helens has available capacity from their Ranney Collector Well and treatment plant. The St. Helens Ranney Well is under the influence of the Columbia River, where the City's proposed Ranney Well maybe under the influence of the Multnomah Channel. An interconnection with St. Helens would allow both cities to benefit from economies of scale in operating the source. However, a long transmission main, between 5.5 and 10 miles, would be required to connect with St. Helens. This transmission main provides a single point of failure, where there is no redundancy should the transmission main be out-of-service due to maintenance, accident, seismic activity, or other failure conditions.

It is the recommended the City investigate the feasibility and cost of this source:

- Confirm the feasibility and cost of constructing the transmission main:
  - Inquire with Northwest Natural Gas on the cost to construct and finance the main.
  - Confirm the ability to place the main in US Highway or Railroad Road ROW.
- Investigate supply availability and cost with St. Helens. This may include a capital buy-in for existing infrastructure and additional charges for new infrastructure.
- Headloss in the transmission main will likely require additional pumping. Identify the location and capacity of the pumping station, where cost will vary depending on if it is located at the St. Helens treatment plant or at a new site in the City.

4.6.3 Supply Options Summary

The City will need to more than double their existing supply capacity, requiring development of new supplies. Using the selected supply options, the next section presents long-term supply alternatives that combine multiple supplies to provide realistic supply portfolios.

Action items to confirm feasibility and refine costs were discussed for each future supply option. Short-term action items to address are summarized at the end of this chapter, in Table 4.23.

4.7 Alternatives Analysis

A combination of supplies can be used to meet supply deficiencies. The four alternatives developed represent the range of supply options:

- Alternative 1: Preserve Existing Surface Water.
- Alternative 2: Groundwater Expansion Only.

- Alternative 3: New Surface Water Source.
- Alternative 4: Interconnection with St. Helens.

The following sections provide a description of the supplies that make up each alternative, the timing of supply developments, a cost estimate, and a discussion of the cost sensitivity.

4.7.1 Alternative 1: Preserve Existing Surface Water

Alternative 1: Preserve Existing Surface Water Supply continues long-term use of the existing surface water supply at its current reliable capacity of 0.36 mgd during the MDD:

- In the short-term, life-safety improvements, seismic improvements, and other repairs will be needed at both Keys Road facilities and the Miller Road WTP.
- Long-term, the Keys Road surface water treatment will be replaced with a new 1 mgd conventional treatment plant.

Future MDD will be met through additional groundwater wells at Dutch Canyon and Miller Road. The Miller Road treatment plant will be expanded to treat additional groundwater supplies. A summary of Alternative 1 costs and components is presented in Figure 4.8.

ALTERNATIVE 1: PRESERVE EXISTING SURFACE WATER

TOTAL COST: $32.3 MILLION

EXISTING SURFACE WATER SUPPLIES

SUPPLY:

0.36 MGD

TRANSMISSION:
25,000 FEET OF 12" DIA. PIPELINE and 4,200 FEET OF 8" DIA. PIPELINE

2 MGD

TREATMENT:
REPLACE WITH NEW 1MGD CONVENTIONAL TREATMENT PLANT

DECOMISSIONED

1 MGD

1 MGD

TOTAL CAPACITY:

EXISTING: 0.36 MGD / FUTURE: 0.36 MGD

COST:

Type	Name	Cost
Transmission	Replace SW Raw Water Transmission	$7.7M
Treatment	Keys Road WTP Rehab	$1.1M
Treatment	SW WTP at Keys	$3.8M
Total		**$12.6M**

NEW DUTCH CANYON WELL

SUPPLY:
ADDITIONAL DUTCH CANYON WELLS

0.48 MGD

0.65 MGD

TRANSMISSION:

1.1 MGD

TREATMENT:
REHAB AND COVER FILTER, SPARE PARTS AT GS FILTER

1.1 MGD

TOTAL CAPACITY:

EXISTING: 0.48 MGD / FUTURE 1.1 MGD

COST:

Type	Name	Cost
Supply	Dutch Canyon Well #2 & #3	$2.6M
Treatment	Keys Road GW WTP	$0.6M
Total		**$3.2M**

NEW MILLER ROAD WELLS

SUPPLY:
ADDITIONAL MILLER RD WELLS

0.65 MGD

1.8 MGD

TRANSMISSION:

1.1 MGD

0.7 MGD

TREATMENT:
REHAB AND EXPAND (1 NEW FILTER)

0.9 MGD

1.7 MGD

TOTAL CAPACITY:

EXISTING: 0.65 MGD / FUTURE 2.7 MGD

COST:

Type	Name	Cost
Supply	Miller Road Well #4 - #8	$8.3M
Transmission	Miller Road RW Transmission	$5.2M
Treatment	Miller Road WTP Rehab	$0.8M
Treatment	Miller Road WTP 0.9 MGD Expansion	$2.2M
Total		**$16.5M**

NEW RANNEY COLLECTOR WELL

SUPPLY:
N/A

TRANSMISSION:
N/A

TREATMENT:
N/A

NOTES:

GW: GROUNDWATER

SW: SURFACE WATER

RW: RAW WATER

All project costs are high-level conceptual costs for alternatives analysis. See Chapter 8 - Capital Improvement Program for up-to-date project budget estimates.

LEGEND:

GREEN SHORT-TERM

RED LONG-TERM

SEISMIC IMPROVEMENTS

LIFE-SAFETY IMPROVEMENTS

REPAIR AND REPLACE IMPROVEMENTS

FIGURE 4.8

SCAPPOOSE WSMP
CITY OF SCAPPOOSE

carollo

4.7.1.1 Supply Timing

The timing of supply developments is presented in Figure 4.9.

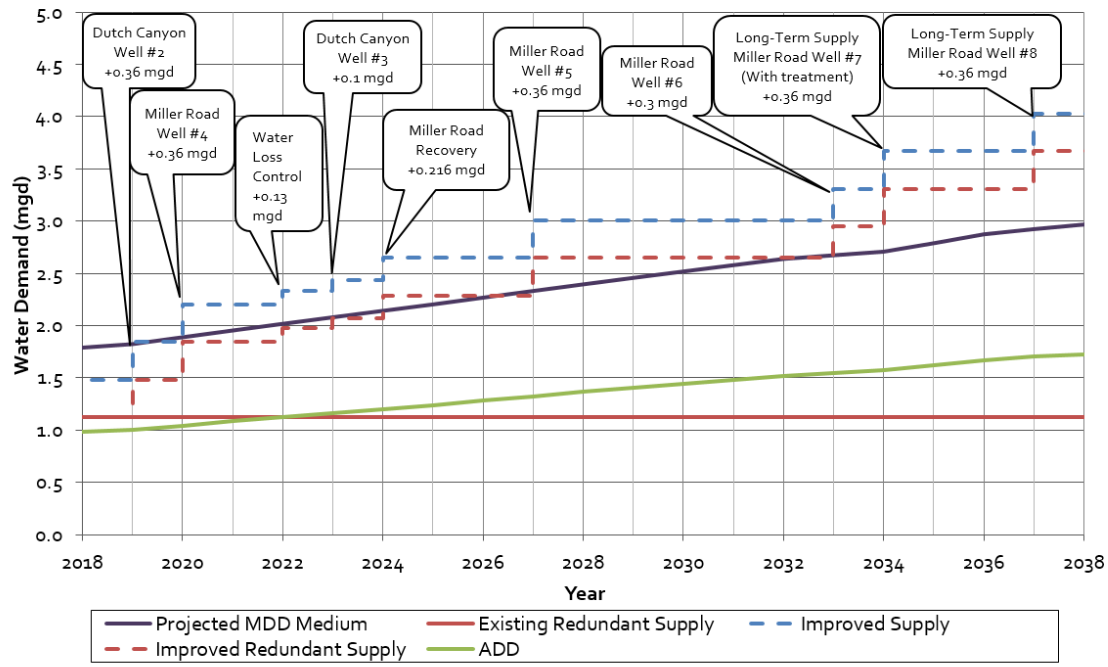


Figure 4.9 Alternative 1 Supply Timing

In order to reliably meet demands, the City will need to select well sites and develop new wells through 2033. Current Miller Road wells utilize 0.65 mgd of the existing treatment plant's 1.83 mgd total capacity. A total of 1.3 mgd groundwater supply can be developed before Miller Road WTP must be expanded, this covers the three short-term Miller Road Wells #4 through #6 and the Miller Road rehabilitation. Long-term supplies developed past 2034, Miller Road Wells #7 and #8, will need an expansion of Miller Road WTP.

4.7.1.2 Cost Estimate

Costs for Alternative 1 are broken down by supply, transmission, and treatment. Costs for individual project components were detailed in Table 4.6 through Table 4.10. Costs were broken out into short-term and long-term.

Short-Term Costs

The short-term supply strategy consists of developing Dutch Canyon Well #2, four additional wells, and rehab efforts at existing facilities. Miller Road Wells #4 through #6 are assumed to have 1,500 LF of 8 inch transmission main. Short-term costs are broken down by supply, transmission, and treatment in Table 4.17. Total short-term supply cost is estimated to be $11.4 million.

Table 4.17 Short-Term Supply Cost Estimate

Cost Type	Item	Cost (millions)
Supply		
	Dutch Canyon Wells #2 and #3.	$2.6
	Supply Miller Road Wells #4 - #6.	$4.9
Transmission		
	Miller Road Well Raw Water Mains.	$1.3
Treatment		
	Keys Road Surface Water WTP Rehab.	$1.1
	Keys Road Groundwater WTP Rehab.	$0.6
	Miller Road WTP Rehab.	$0.8
	Total Short-Term Cost	**$11.3**

Long-Term Costs

Long-term costs include costs for preserving the existing surface water supply and developing additional groundwater supplies, and are summarized in Table 4.18. The existing surface water treatment plant will be replaced with new conventional treatment. Miller Road Wells #7 and #8, with Miller Road Well #7 constructed with a 10,000 LF transmission main to Miller Road. An expansion of Miller Road WTP will be needed with the additional Miller Road wells. The total estimated long-term cost is estimated to be $21.0 million, or $29.20 per gallon. Total cost for Alternative 1 is estimated to be $32.3 million.

Table 4.18 Alternative 1 Cost Estimate

Cost Type	Item	Cost (millions)
Supply		
	Supply Miller Road Wells #7 - #8.	$3.4
Transmission		
	Replace Surface Water Raw Water Transmission Mains to Keys Road WTP.	$7.7
	Miller Road Well Raw Water Mains.	$3.9
Treatment		
	New 1.0 mgd Surface Water Treatment at Keys Road.	$3.8
	Miller Road WTP 0.9 mgd Expansion.	$2.2
	Long-Term Cost	**$21.0**
	Short-Term Cost	**$11.3**
	Total Cost	**$32.3**
	Long-Term Supply Cost (0.72 mgd of New Supply)	**$29.20 / gallon**

4.7.1.3 Cost Sensitivity

Of the action items detailed in Table 4.23, the following were assumed to have the greatest impact on the cost sensitivity for Alternative 1:

- Install monitoring to quantify the surface water supply capacity. Should the City be able to access greater than 0.36 mgd (250 gpm) reliably from their surface water source, maintaining the surface water supply will be more cost-effective on a per gallon basis.
- Identify yield of new wells. If well yields greater than 0.36 mgd can be achieved, fewer wells will need to be developed.
- Identify water quality of new wells. High iron and manganese concentrations will require more treatment plant capacity or may eliminate well locations from consideration.
- Identify locations for new Miller Road wells. Raw Water transmission costs vary depending on location. Miller Road Wells #7, #8, and #9 are assumed to be located at the school complex, requiring 10,000 feet of raw water main from the wells to the treatment plant. Shorter raw water transmission main lengths can reduce costs by up to $2.5 million. Land acquisition costs may vary.

4.7.2 Alternative 2: Groundwater Expansion Only

4.7.2.1 Overview

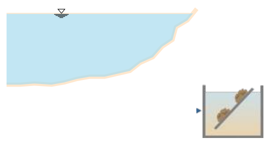
Alternative 2 addresses the City's future supply needs through expansion of groundwater supplies. The City will develop an additional 2.4 mgd of groundwater supplies beyond Dutch Canyon Well #2. As part of this alternative, the City would abandon their existing surface water supply once sufficient supplies have been developed. Assuming a 0.36 mgd well yield, the City will need to develop a third Dutch Canyon well and six additional wells near Miller Road. This would require the City expand the existing Miller Road WTP with two new filters to treat the new sources. The existing surface water was assumed to be maintained in the short-term and decommissioned once long-term supplies were developed. A summary of the components and associated costs is presented in Figure 4.10.

ALTERNATIVE 2: GROUNDWATER EXPANSION ONLY

TOTAL COST: $24.7 MILLION

EXISTING SURFACE WATER SUPPLIES

SUPPLY:



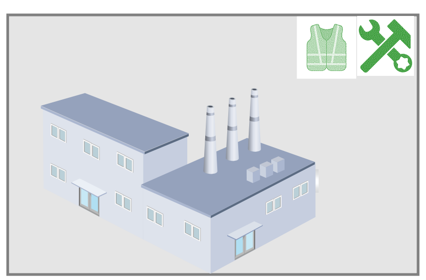
0.36 MGD

TRANSMISSION:

2 MGD

TREATMENT:

LIFE-SAFETY AND REHAB ON EXISTING, DECOMMISSION FACILITY IN LONG-TERM



1 MGD

TOTAL CAPACITY:

EXISTING: 0.36 MGD / FUTURE 0 MGD

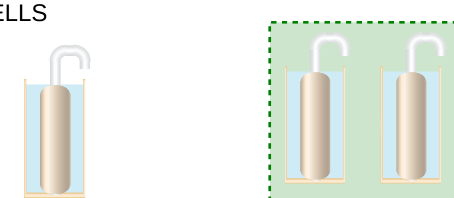
COST:

Type	Name	Cost
Treatment	Keys Road WTP Rehab	$1.1M
Total		**$1.1M**

NEW DUTCH CANYON WELL

SUPPLY:

ADDITIONAL DUTCH CANYON WELLS



0.48 MGD

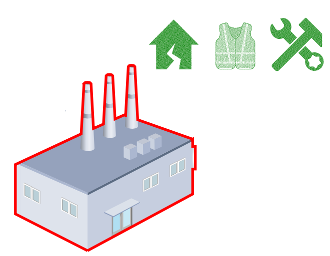
0.65 MGD

TRANSMISSION:

1.1 MGD

TREATMENT:

REHAB AND COVER FILTER, SPARE PARTS AT GS FILTER



1.1 MGD

TOTAL CAPACITY:

EXISTING: 0.48 MGD / FUTURE 1.1 MGD

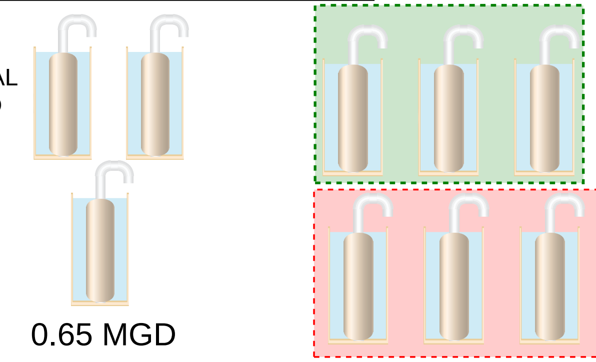
COST:

Type	Name	Cost
Supply	Dutch Canyon Well #2 & #3	$2.6M
Treatment	Keys Road GW WTP	$0.6M
Total		**$3.2M**

NEW MILLER ROAD WELLS

SUPPLY:

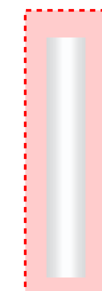
ADDITIONAL MILLER RD WELLS



0.65 MGD

2.1 MGD

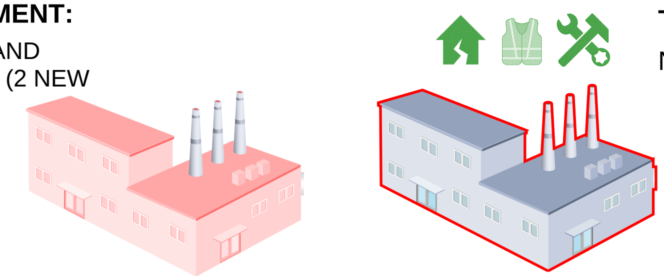
TRANSMISSION:



2.1 MGD

TREATMENT:

REHAB AND EXPAND (2 NEW FILTER)



1.8 MGD

1.7 MGD

TOTAL CAPACITY:

EXISTING: 0.65 MGD / FUTURE 2.8 MGD

COST:

Type	Name	Cost
Supply	Miller Road Wells #4 - #9	$9.9M
Transmission	Miller Road RW	$5.5M
Treatment	Miller Road WTP Rehab	$0.8M
Treatment	Miller Road GW WTP 1.8 MGD Expansion	$4.2M
Total		**$20.4M**

NEW RANNEY COLLECTOR WELL

SUPPLY:

N/A

TRANSMISSION:

N/A

TREATMENT:

N/A

NOTES:

GW: GROUNDWATER

SW: SURFACE WATER

RW: RAW WATER

All project costs are high-level conceptual costs for alternatives analysis. See Chapter 8 - Capital Improvement Program for up-to-date project budget estimates.

LEGEND:

GREEN SHORT-TERM

RED LONG-TERM

SEISMIC IMPROVEMENTS

LIFE-SAFETY IMPROVEMENTS

REPAIR AND REPLACE IMPROVEMENTS

FIGURE 4.10

SCAPPOOSE WSMP
CITY OF SCAPPOOSE

4.7.2.2 Supply Timing

The timing and magnitude of supply developments is shown in Figure 4.11.

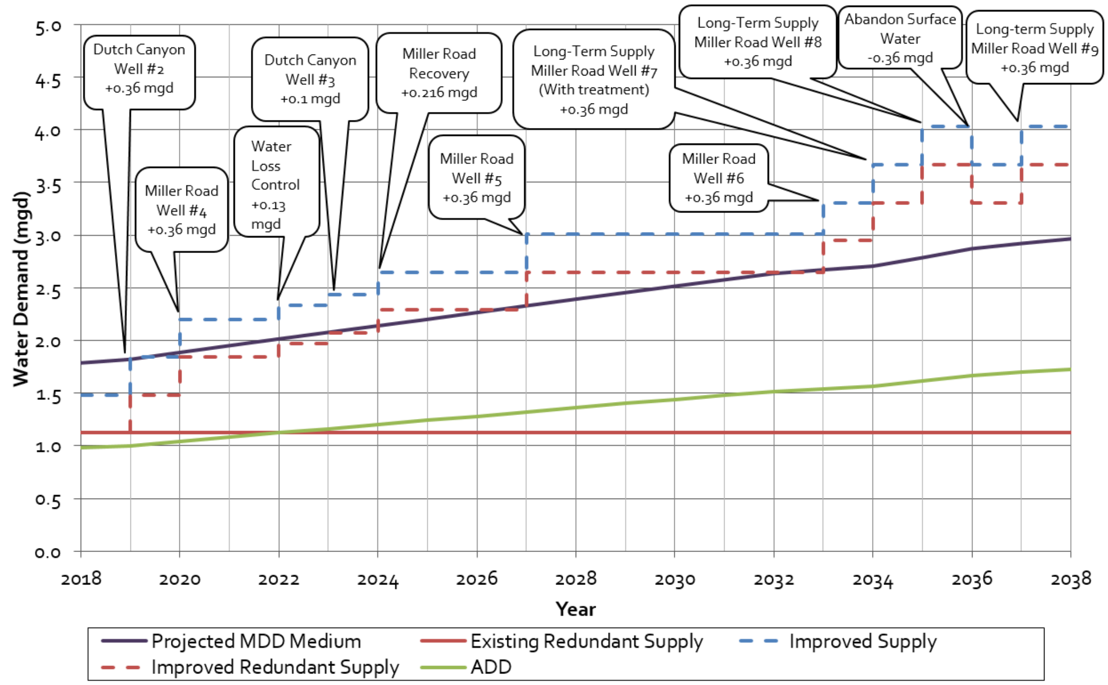


Figure 4.11 Alternative 2 Supply Timing

Similar to Alternative 1, the City will need to develop new groundwater supplies through 2033 to meet projected growth in demand. Expansion of Miller Road WTP will be needed by the time additional Miller Road wells are developed in 2034. Once sufficient supplies have been developed by 2035, the City can decommission their surface water supply.

4.7.2.3 Cost Estimate

Long-term supply costs for Alternative 2 are broken down by supply, transmission, and treatment in Table 4.19. Short-term supply development costs were summarized in Table 4.17. Miller Road Wells #4 through #6 are assumed to have 1,500 LF of 8 inch transmission main. Miller Road Wells #7 through #9 were assumed to be located at the schools, with Miller Road Well #7 constructed with a 10,000 LF transmission main to Miller Road. Expansion of Miller Road WTP will be needed to treat Miller Road Wells #7 through #9. Long-term supply costs are estimated to be $13.4 million, or $12.40 per gallon for 1.08 mgd of new supply. The total estimated cost for Alternative 2 is $24.7 million.

Table 4.19 Alternative 2 Cost Estimate

Cost Type	Item	Cost (millions)
Supply		
	Supply Miller Road Wells #7 - #9.	$5.0
Transmission		
	Miller Road Well Raw Water Mains.	$4.2
Treatment		
	Miller Road WTP 1.8 mgd Expansion.	$4.2
	Long-Term Cost	$13.4
	Short-Term Cost	$11.3
	Total Cost	$24.7
	Long-Term Supply Cost (1.08 mgd of New Supply)	$12.40 / gallon

4.7.2.4 Cost Sensitivity

Of the action items presented in Table 4.23, the following have the most impact on the costs for Alternative 2:

- Identify locations for new Miller Road wells. Raw Water transmission costs vary depending on location. Miller Road Wells #7, #8, and #9 are assumed to be located at the school complex, requiring 10,000 feet of raw water main from the wells to the treatment plant. Shorter raw water transmission main lengths can reduce costs by up to $2.5 million. Identify yield of new wells. If well yields greater than 0.36 mgd can be achieved, fewer wells will need to be developed. Land acquisition cost may vary.
- Identify water quality of new wells. High Iron and Manganese concentrations will require more treatment plant capacity or may eliminate well locations from consideration.

4.7.3 Alternative 3: New Surface Water Source

4.7.3.1 Overview

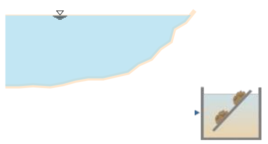
Alternative 3 relies on the City developing a new surface water source to meet their long-term supply needs. A new 1.1 mgd Ranney well and treatment plant (expandable to 3.0 mgd) will be constructed. An additional 1.1 mgd of groundwater supply from wells near Miller Road and a third Dutch Canyon Well provide supply while the Ranney collector well is developed. In the long-term, the City will abandon their existing surface water supply. A summary of the Alternative 3 components and costs is presented in Figure 4.12.

ALTERNATIVE 3: NEW SURFACE WATER SOURCE

TOTAL COST: $23.7 MILLION

EXISTING SURFACE WATER SUPPLIES

SUPPLY:

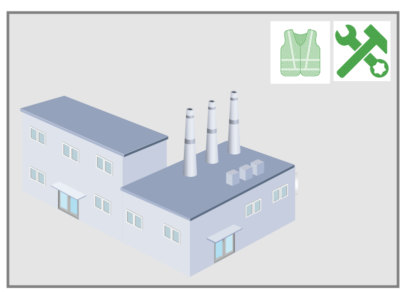


0.36 MGD

TRANSMISSION:

2 MGD

TREATMENT:
LIFE-SAFETY AND REHAB ON EXISTING, ABANDON FACILITY IN LONG-TERM



1 MGD

TOTAL CAPACITY:

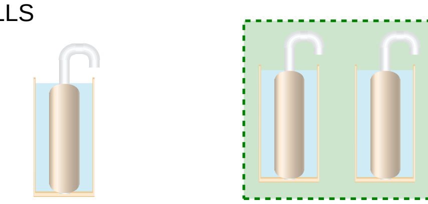
EXISTING: 0.36 MGD / FUTURE 0 MGD

COST:

Type	Name	Cost
Treatment	Keys Road WTP Rehab	$1.1M
Total		**$1.1M**

NEW DUTCH CANYON WELL

SUPPLY:
ADDITIONAL DUTCH CANYON WELLS

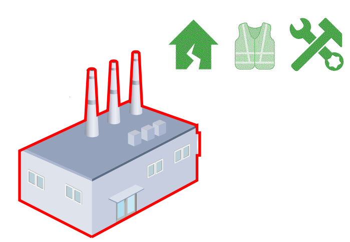


0.48 MGD 0.65 MGD

TRANSMISSION:

1.1 MGD

TREATMENT:
REHAB AND COVER FILTER, SPARE PARTS AT GS FILTER



1.1 MGD

TOTAL CAPACITY:

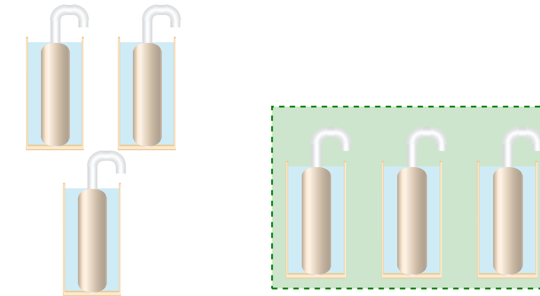
EXISTING: 0.48 MGD / FUTURE 1.1 MGD

COST:

Type	Name	Cost
Supply	Dutch Canyon Well #2 & #3	$2.6M
Treatment	Keys Road GW WTP	$0.6M
Total		**$3.2M**

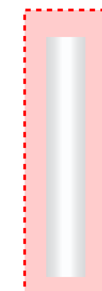
NEW MILLER ROAD WELLS

SUPPLY:
ADDITIONAL MILLER RD WELLS



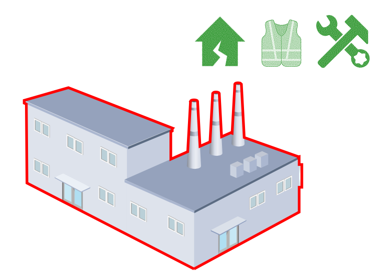
0.65 MGD 1 MGD

TRANSMISSION:



1 MGD

TREATMENT:
LIFE-SAFETY AND REHAB ON EXISTING



1.7 MGD

TOTAL CAPACITY:

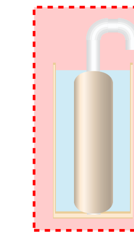
EXISTING: 0.65 MGD / FUTURE 1.7 MGD

COST:

Type	Name	Cost
Supply	Miller Road Well #4 - #6	$4.9M
Transmission	Miller Road RW	$1.3M
Treatment	Miller Road WTP Rehab	$0.8M
Total		**$7.0M**

NEW RANNEY COLLECTOR WELL

SUPPLY:
NEW RANNEY COLLECTOR + PUMP HOUSE

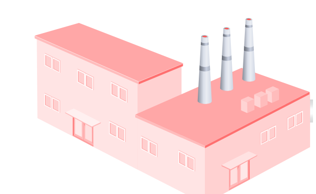


1 MGD

TRANSMISSION:

3.4 MGD

TREATMENT:



1.1 MGD

TOTAL CAPACITY:

FUTURE 1.08 MGD

COST:

Type	Name	Cost
Wells	Ranney Collector	$4.3M
	Ranney Well RW	
Transmission	Transmission	$2.7M
Treatment	SW WTP at Miller Road	$5.4M
Total		**$12.4M**

NOTES:

GW: GROUNDWATER

SW: SURFACE WATER

RW: RAW WATER

All project costs are high-level conceptual costs for alternatives analysis. See Chapter 8 - Capital Improvement Program for up-to-date project budget estimates.

LEGEND:

GREEN SHORT-TERM

RED LONG-TERM

SEISMIC IMPROVEMENTS

LIFE-SAFETY IMPROVEMENTS

REPAIR AND REPLACE IMPROVEMENTS

carollo

FIGURE 4.12

SCAPPOOSE WSMP
CITY OF SCAPPOOSE

4.7.3.2 Timing

The timing and magnitude of supply improvements is shown in Figure 4.13.

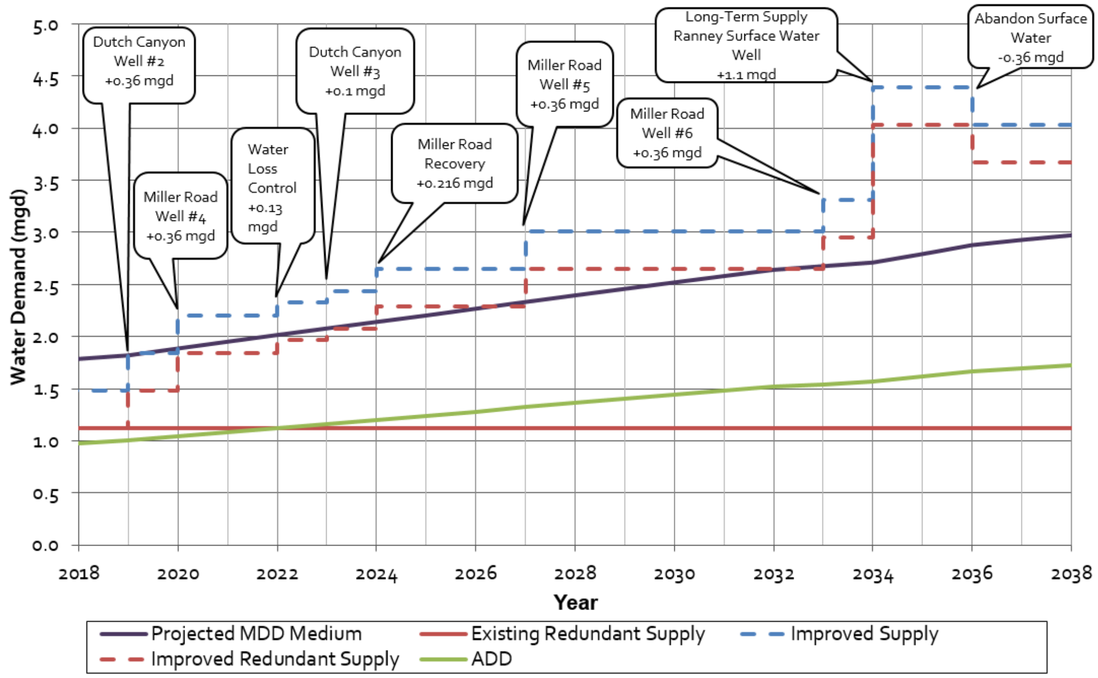


Figure 4.13 Alternative 3 Supply Timing

The City will need to develop additional groundwater wells through 2033 to ensure supplies reliably meet projected demands. This is consistent with Alternatives 1 and 2. Should the City achieve greater well yields, the timing and number of supplies may change. The Ranney collector well should be developed and brought online by 2034, after which time the City can decommission their existing surface water facility.

4.7.3.3 Cost Estimate

Short-term costs for all four alternatives were presented in Table 4.17. Costs for Alternative 3 long-term supplies are broken down by supply, transmission, and treatment in Table 4.20. Long-term supply for Alternative 3 consists of developing a Ranney well at Chapman Landing with an initial 1.1 mgd capacity, a new advanced surface water treatment facility, and 7,300 LF of transmission main from the Ranney well to the treatment plant. Total long-term supply cost is estimated to be $12.4 million, or $11.50 per gallon of new supply. The total cost for Alternative 3 is estimated to be $23.7 million.

Table 4.20 Alternative 3 Cost Estimate

Cost Type	Item	Cost (millions)
Supply		
	Ranney Collector Well.	$4.3
Transmission		
	Ranney Well Raw Water Transmission Main.	$2.7
Treatment		
	New Advanced Surface Water Treatment Plant at Miller Road.	$5.4
	Long-Term Cost	**$12.4**
	Short-Term Cost	**$11.3**
	Total Cost	**$23.7**
	Long-Term Supply Cost (1.08 mgd of New Supply)	**$11.50 / gallon**

4.7.3.4 Cost Sensitivity

Of the action items presented in Table 4.23, the following have the most impact on the costs for Alternative 3:

- Finalize well location and expected yield. Well location will determine transmission length and costs, potentially impacting the magnitude and timing of Ranney Well supplies.
- Identify Ranney Well costs and yield may vary based on the hydrogeology of the Chapman landing area.
- Determine surface water quality. Water quality will dictate treatment costs.

4.7.4 Alternative 4: Interconnection with St. Helens

Alternative 4 addresses future supply needs primarily through interconnection with St. Helens water supply. In the short-term, the City should develop additional groundwater wells to meet supply needs. This allows the City to meet supply requirements while negotiating with St. Helens and to construct the infrastructure needed. The transmission main from St. Helens to Scappoose is assumed to be built by NW Natural.

The timing of supply developments is shown in Figure 4.14. Similar to other alternatives presented, additional groundwater supplies are developed through 2033 to meet short-term supply needs. All components of St. Helens interconnection are assumed to be constructed and buy-in agreements complete by 2034. The City can then decommission the existing surface water facility after this is complete.

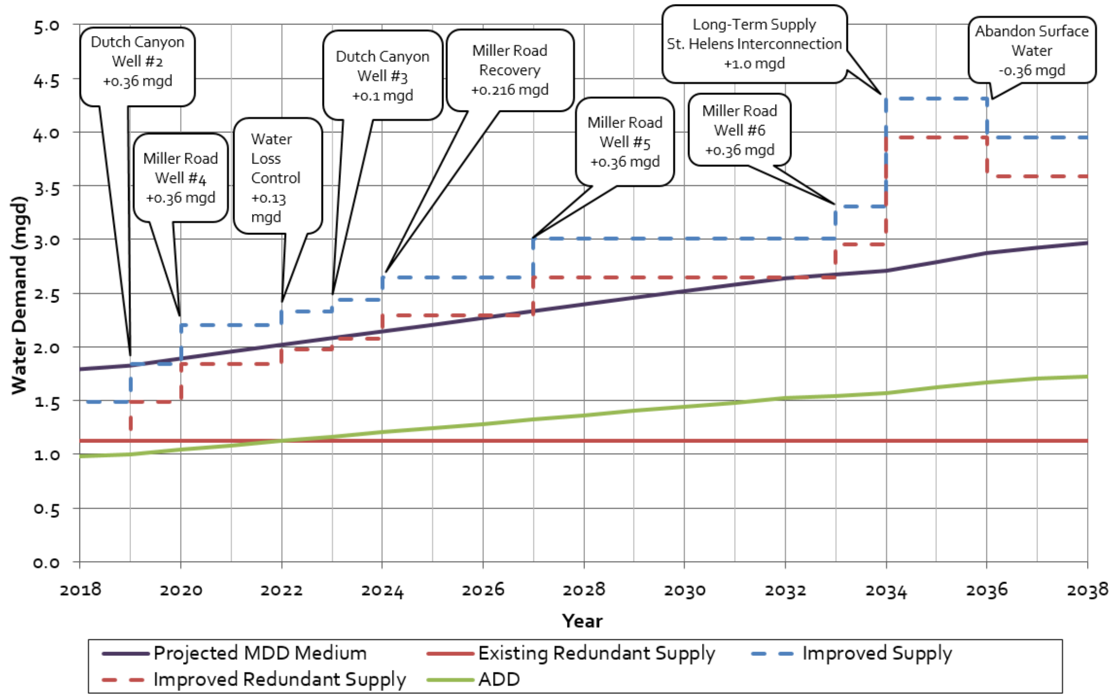


Figure 4.14 Alternative 4 Supply Timing

4.7.4.1 Cost Estimate

Short-term supply costs are consistent across all four alternatives and were summarized previously in Table 4.17. Long-term supply costs and total costs for Alternative 4 are summarized in Table 4.21. Costs include the assumed costs of $5.4 million for supply buy-in and $3.9 million for treatment buy-in. The construction of the transmission main and BPS are also included in the cost estimate. Refinement of costs for this alternative will require negotiation with St. Helens regarding capital buy-in costs and volumetric rates. The City will also need to work with NW Natural to determine the costs for transmission. Transmission costs were assumed to be for 8.5 miles of 24 inch main from St. Helens' existing filtration plant. Long-term supply costs are estimated to be $33.6 million, or $31.10 per gallon. The total estimated cost for Alternative 4 is $44.9 million.

4.7.4.2 Cost Sensitivity

The City should work with NW Natural and St. Helens to refine the long-term supply cost estimates.

Table 4.21 Alternative 4 Cost Estimate

Cost Type	Item	Cost (millions)
Supply		
	St. Helens Supply Buy-In.	$3.9
Transmission		
	45,000 LF 24-inch Transmission Main from St. Helens.	$21.6
	1 mgd Pump Station from St. Helens.	$1.1
	Pump Station Expansion to 3 mgd Capacity.	$1.6
Treatment		
	St. Helens Treatment Capacity Buy-In.	$5.4
	Long-Term Cost	**$33.6**
	Short-Term Cost	**$11.3**
	Total Cost	**$44.9**
	Long-Term Supply Cost (1.08 mgd of New Supply)	**$31.10 / gallon**

4.7.5 Recommended Alternative

Alternatives were developed from available future supply sources. Each of the four alternatives require construction of additional groundwater wells in the short-term that make use of existing water rights and treatment facilities. Long-term, the City can continue to develop groundwater supplies (Alternative 2). Alternatively, the City can develop a Ranney Well on the Multnomah Channel (Alternative 3) or construct an intertie with St. Helens (Alternative 4). Due to the length of conveyance required, the City-owned Ranney well is anticipated to be less expensive. The City's existing surface water supplies require expensive R&R; making long-term use of the supply costly. Alternative 2 - Groundwater Expansion Only and Alternative 3 - Ranney Well have similar costs, between $23.7 million and $24.7 million. Alternatives 1 and 4 have higher costs of $32.3 million to $44.9 million. Therefore, it is recommended the City budget $11.3 million for the following costs in the short-term, as the projects were included in all supply alternatives:

Table 4.22 0 – 20 Years Project Planning

Year Implemented	New Supply Sources Daily Demand	Reliable Withdrawal		New Withdrawal		Total Cumulative Withdrawal	
		mgd	cfs	mgd	cfs	mgd	cfs
	Existing Reliable Supply	1.12	1.73			1.12	1.73
2019	Dutch Canyon Well #2			0.36	0.56	1.48	2.29
2020	Miller Road Well #4			0.36	0.56	1.84	2.85
2022	Water Loss Control (3)			0.13	0.20	1.97	3.05
2023	Dutch Canyon Well #3(2)			0.10	0.15	2.07	3.20
2024	New Miller Road Well Recovery(1)			0.22	0.34	2.29	3.54
2027	Miller Road Well #5			0.36	0.56	2.65	4.10

Year Implemented	New Supply Sources Daily Demand	Reliable Withdrawal		New Withdrawal		Total Cumulative Withdrawal	
		mgd	cfs	mgd	cfs	mgd	cfs
2033	Miller Road Well #6			0.3	0.46	2.95	4.56
2034	New Supply			1.02	1.58	3.97	6.14
	Supply Total					3.97	6.14
	Demand (MDD)					2.97	4.60
	Excess supply for future growth					1.0	1.54

Notes:

(1) City has observed a decline in yield from its Miller Road wells. The City cleaned the wells and conducted maintenance on the well pumps, but has not restored the full yield from the wells. Permit G-17644 provides seven well withdrawal locations to provide the capacity for full use of the water right.

(2) City has observed a decline in yield from Dutch Canyon Well No. 1. The City is working with a hydrogeologist to determine next steps as part of the ongoing Dutch Canyon Well No. 2 construction project.

(3) The water loss control program "supply" equates to maintaining a water loss of approximately 10 percent through the end of 20-year period. This includes the initial 0.13 MG (0.20 cfs) reduction of real losses from the 2-year Water Loss Control program which is anticipated to reduce the system to 10 percent water loss.

(4) Years are for planning purposes only, so differences in yield will change the timing of projects.

It is recommended that the City budget an additional $12.4 million for long-term supply, effectively the cost for Alternative 3 beyond 2033. There are unknowns in all alternatives that may cause large changes in costs. Action items to confirm supply effectiveness and refine costs of the supply options are presented in Table 4.23. Action items to address are broken down by timing, with items to address within the next two years and the next two to five years. Addressing these action items will further aid the City in making difficult future supply decisions. As the City completes the activities, they should revise the yield, water quality, and costs assumptions of this Chapter.

Table 4.23 Summary of Action Items for Developing Future Supplies

Supply Option	0 – 2 Years	2 – 5 Years
Water Loss Control Plan	Implement Water Loss Control Plan	Re-evaluate activities if water loss is not reduced to 10 percent.
Existing Surface Water Supplies	Measure streamflow at existing diversion structures.	Perform leak detection on raw water transmission mains. Install pigging ports for sediment removal. Perform life-safety and other rehab improvements to existing Keys Road surface water facility.
New Miller Road Wells	Drill test well at MP-1 site.	Acquire property and drill test well for CZ-1 well. Drill test well for high school/elementary school wells. Perform life-safety, seismic, and other repair improvements to the existing Miller Road WTP.
New Dutch Canyon Well	Finalize combined production capacity from existing Dutch Canyon wells.	Acquire property for third well site. Perform life-safety, seismic, and other repair improvements to the existing Keys Road groundwater treatment. Investigate construction of a high-rate iron and manganese treatment system at Dutch Canyon site.
New Ranney Collector Well		Drill a test well to determine hydrogeological feasibility. Perform water quality sampling for test well and in Multnomah Channel.
Interconnection with St. Helens		Determine buy-in and other costs associated with the existing St. Johns Ranney Well. Work with NW Natural to identify transmission project costs and ROW. Determine buy-in and other costs associated with water treatment.

Chapter 5

WATER QUALITY AND TREATMENT

5.1 Introduction

This chapter assesses the City of Scappoose's (City's) raw and finished water quality, current and anticipated water quality regulations and compliance history, water treatment plant performance, and recommends future repair and replacement-related capital improvements. Specifically, this chapter:

- Summarizes the raw and finished and raw water quality in the Miller Road and Keys Road Water Treatment Plant (WTP).
- Discusses current and anticipated water quality regulations.
- Summarizes the current treatment process types, capacity, overall performance, and treatment system conditions.
- Identifies improvements to the City's water treatment systems that are required to meet current and future regulations.
- Identifies equipment repair and replacement improvements required to ensure continued compliance with drinking water regulations.
- Identifies recommended studies to identify seismic and life-safety improvements, as well as optimize overall plant performance.

NOTE: Capacity improvement analysis and associated costs are presented in Chapter 4 and will not be discussed in this chapter.

5.2 Water Quality Level of Service

The City is committed to providing safe and reliable drinking water to its customers; complying with all federal drinking water regulations, as adopted by the State of Oregon. Further, the City's goal is to removal of iron from its groundwater to levels established through secondary limits by the State of Oregon.

5.3 Water Quality & Regulatory Compliance

5.3.1 Water Quality Monitoring

The City's Water Quality Monitoring Program complies with federal drinking water regulations as adopted by the State of Oregon. The City installed water quality instrumentation on its groundwater and surface water treatment plants to record daily trends for temperature, pH, turbidity, alkalinity, and iron. State-certified laboratory analysis is performed to supplement this 'process' data in compliance with the drinking water quality regulations.

The following sections summarize the results from these monitoring efforts, and discuss the City's compliance with Oregon Health Authority (OHA) requirements.

5.3.1.1 Raw Water Quality

The City currently receives raw water from three sources: Miller Road groundwater, Ditch Canyon groundwater, and Keys Road surface water. Keys Road surface water source is located at South Fork Scappoose Creek, Gourley Creek, and Lazy Creek.

Table 5.1 presents range, average, and percentages for several raw water quality parameters in each of the entry points at Miller Road and Keys Road collected between May 2007 and June 2017. As shown, Keys Road's wells typically had less iron than the wells at Miller's Road.

5.3.1.2 Finished Water Quality

Table 5.2 presents range, average, and percentile values for several finished water quality parameters at Miller Road WTP and Keys Road WTP collected between May 2007 and June 2017. For the Keys Road WTP, finished water quality data combines both groundwater and surface water source. As shown, the greensand filters were excellent at removing iron, and chlorine and fluoride residual levels were consistently at a minimum of 0.2 milligrams per liter (mg/L) throughout testing.

Table 5.1 Summary of City's Raw Water Quality from May 2007 through June 2017

Contaminant	Unit(1)	Miller Road Groundwater					Keys Road Groundwater					Keys Road Surface Water				
		Range	Average	Percentile			Range	Average	Percentile			Range	Average	Percentile		
				5th	50th	95th			5th	50th	95th			5th	50th	95th
Turbidity	NTU	0.01 - 5.0	0.21	0.06	0.16	0.51	0.92 - 14.9	4.7	1.9	4.6	7.3	0.09 - 18.7	2.5	0.92	2.1	5.4
pH	-	6.2 - 8.4	7.22	6.7	7.3	7.5	4.1 - 8.0	6.91	6.3	7	7.2	5.0 - 8.2	7.2	6.5	7.3	7.6
Temperature	°C	11.8 - 19.2	13.4	12.6	13.3	14.8	10.0 - 21.3	12.81	10.5	12.8	15.3	5.4 - 22.5	13	7.6	13.4	17.3
Iron (Fe)	mg/L	0.08 - 1.8	1.17	1	1.2	1.3	0.03 - 3.4	0.45	0.26	0.43	0.68	-	-	-	-	-
Alkalinity	mg/L	-	-	-	-	-	-	-	-	-	-	7.0 - 36.0	20.5	9	21	29.7

Note:
(1) NTU: nephelometric turbidity unit °C: degree(s) Celsius

Table 5.2 Summary of City's Finished Water Quality and Corresponding Finished Water MCL Collected from March 2007 through September 2017

Contaminant	Unit[1]	Finished Water MCL[1]	Keys Road WTP					Miller Road WTP				
			Range	Average	Percentile			Range	Average	Percentile		
					5th	50th	95th			5th	50th	95th
General												
Turbidity	NTU		0.02 - 0.29	0.05	0.02	0.04	0.09	0 - 0.27	0.03	0.01	0.03	0.05
Total Organic Carbon	mg/L											
Alkalinity	mg/L as $CaCO_3$	None										
Temperature	°C		8.4 - 19.0	12.9	9.1	12.8	16.4	7.3 - 18.7	13.2	12.5	13.1	14.2
Secondary Contaminants												
Color	-	15 color units										
Corrosivity	-	Non-corrosive										
Foaming Agents	mg/L	0.5										
pH	-	6.5-8.5	6.9 - 9.1	7.6	7.2	7.5	8.2	6.8 - 8.7	7.4	7.2	7.4	7.8
Hardness	mg/L as $CaCO_3$	250										
Odor	-	3 TON										
Total Dissolved Solids	mg/L	500										
Aluminum	mg/L	0.05-2.0										
Chloride	mg/L	250										
Fluoride	mg/L	4	0.1 - 1.9	0.51	0.13	0.47	0.97	0.2 - 1.6	0.63	0.28	0.6	1
Iron	mg/L	0.3	0 - 0.13	0.03	0	0.02	0.05	0 - 0.58	0.01	0	0.01	0.05
Manganese	mg/L	0.05										
Silver	mg/L	0.1										
Sulfate	mg/L	250										
Zinc	mg/L	5										
Contaminant	Unit	Finished Water MCL	No. of samples	No. of Detects	Min	Max	Average	No. of samples	No. of Detects	Min	Max	Average
Inorganic Contaminants (IOCs)												
Antimony (total)	mg/L	0.006	1	0	-	-	-	2	0	-	-	-
Arsenic	mg/L	0.01	3	0	-	-	-	4	0	-	-	-
Asbestos	MFL	7	-	-	-	-	-	-	-	-	-	-
Barium	mg/L	2	1	0	-	-	-	2	1	-	-	0.0326
Beryllium (total)	mg/L	0.004	1	0	-	-	-	2	0	-	-	-
Cadmium	mg/L	0.005	1	0	-	-	-	2	0	-	-	-
Chromium	mg/L	0.1	1	0	-	-	-	2	0	-	-	-
Copper	mg/L	1.3	-	-	-	-	-	-	-	-	-	-
Cyanide	mg/L	0.2	1	0	-	-	-	2	0	-	-	-
Lead	mg/L	0.015	-	-	-	-	-	1	0	-	-	-
Mercury	mg/L	0.002	1	0	-	-	-	2	0	-	-	-

Table 5.2 Summary of City's Finished Water Quality and Corresponding Finished Water MCL Collected from March 2007 through September 2017 (continued)

Contaminant	Unit	Finished Water MCL	Keys Road WTP					Miller Road WTP				
			Range	Average	Percentile			Range	Average	Percentile		
					5th	50th	95th			5th	50th	95th
Inorganic Contaminants (IOCs) (continued)												
Nickel	mg/L	non-regulated	1	0	-	-	-	2	0	-	-	-
Nitrate	mg/L	10	11	10	0.344	1.1	0.654	12	0	-	-	-
Nitrate-Nitrite	mg/L	10	2	2	0.6	1.1	0.85	4	0	-	-	-
Nitrite	mg/L	1	2	0	-	-	-	4	0	-	-	-
Selenium	mg/L	0.05	1	0	-	-	-	2	0	-	-	-
Sodium	mg/L	non-regulated	1	1	-	-	9.8	2	2	21.5	21.8	21.65
Thallium (total)	mg/L	0.002	1	0	-	-	-	2	0	-	-	-
Synthetic Organic Contaminants (SOCs)												
2,4-D	mg/L	0.07	8	0	-	-	-	6	0	-	-	-
2,4,5-TP	mg/L	0.05	8	0	-	-	-	6	0	-	-	-
Atrazine	mg/L	0.003	8	0	-	-	-	6	0	-	-	-
Benzo(a)pyrene	mg/L	0.0002	8	0	-	-	-	6	0	-	-	-
BHC-Gamma (Lindane)	mg/L	0.0002	8	0	-	-	-	6	0	-	-	-
Carbofuran	mg/L	0.04	8	0	-	-	-	6	0	-	-	-
Chlordane	mg/L	0.002	8	0	-	-	-	6	0	-	-	-
Dalapon	mg/L	0.2	8	0	-	-	-	6	0	-	-	-
1,2-Dibromo-3-chloropropane	mg/L	0.0002	8	0	-	-	-	6	0	-	-	-
DI(2-ethylhexyl) adipate	mg/L	0.4	8	0	-	-	-	6	0	-	-	-
DI(2-ethylhexyl) phthalate	mg/L	0.006	8	0	-	-	-	6	0	-	-	-
Dinoseb	mg/L	0.007	8	0	-	-	-	6	0	-	-	-
Diquat	mg/L	0.02	8	0	-	-	-	6	0	-	-	-
Endothall	mg/L	0.1	8	0	-	-	-	6	0	-	-	-
Endrin	mg/L	0.002	8	0	-	-	-	6	0	-	-	-
Ethylene Dibromide	mg/L	0.00005	8	0	-	-	-	6	0	-	-	-
Glyphosate	mg/L	0.7	8	0	-	-	-	6	0	-	-	-
Heptachlor	mg/L	0.0004	8	0	-	-	-	6	0	-	-	-
Heptachlor Epoxide	mg/L	0.0002	8	0	-	-	-	6	0	-	-	-
Hexachlorobenzene	mg/L	0.001	8	0	-	-	-	6	0	-	-	-
Hexachlorocyclopentadiene	mg/L	0.05	8	0	-	-	-	6	0	-	-	-
Lasso (Alachlor)	mg/L	0.002	8	0	-	-	-	6	0	-	-	-
Methoxychlor	mg/L	0.04	8	0	-	-	-	6	0	-	-	-
Oxamyl	mg/L	0.2	8	0	-	-	-	6	0	-	-	-
Pentachlorophenol	mg/L	0.001	8	0	-	-	-	6	0	-	-	-
Picloram	mg/L	0.5	8	0	-	-	-	6	0	-	-	-

Table 5.2 Summary of City's Finished Water Quality and Corresponding Finished Water MCL Collected from March 2007 through September 2017 (continued)

Contaminant	Unit	Finished Water MCL	Keys Road WTP					Miller Road WTP				
			Range	Average	Percentile			Range	Average	Percentile		
					5th	50th	95th			5th	50th	95th
Synthetic Organic Contaminants (SOCs) (continued)												
Polychlorinated Biphenyls (PCB) (total)	mg/L	0.0005	8	0	-	-	-	6	0	-	-	-
Simazine	mg/L	0.004	8	0	-	-	-	6	0	-	-	-
Toxaphene	mg/L	0.003	8	0	-	-	-	6	0	-	-	-
Volatile Organic Contaminants (VOCs)												
1,1,1-Trichloroethane	mg/L	0.2	11	0	-	-	-	6	0	-	-	-
1,1,2-Trichloroethane	mg/L	0.005	11	0	-	-	-	-	-	-	-	-
o-Dichlorobenzene	mg/L	0.6	11	0	-	-	-	3	0	-	-	-
p-Dichlorobenzene	mg/L	0.075	11	0	-	-	-	3	0	-	-	-
1,2-Dichloroethane	mg/L	0.005	11	0	-	-	-	3	0	-	-	-
1,1-Dichloroethylene	mg/L	0.007	11	0	-	-	-	3	0	-	-	-
cis-1,2-Dichloroethylene	mg/L	0.07	11	0	-	-	-	3	0	-	-	-
trans-1,2-Dichloroethylene	mg/L	0.1	11	0	-	-	-	3	0	-	-	-
1,2-Dichloropropane	mg/L	0.005	11	0	-	-	-	3	0	-	-	-
1,2,4-Trichlorobenzene	mg/L	0.07	11	0	-	-	-	3	0	-	-	-
Benzene	mg/L	0.005	11	0	-	-	-	3	0	-	-	-
Carbon Tetrachloride	mg/L	0.005	11	0	-	-	-	3	0	-	-	-
Chlorobenzene	mg/L	0.1	11	0	-	-	-	3	0	-	-	-
Dichloromethane	mg/L	0.005	11	0	-	-	-	3	0	-	-	-
Ethylbenzene	mg/L	0.7	11	0	-	-	-	3	0	-	-	-
Styrene	mg/L	0.1	11	0	-	-	-	3	0	-	-	-
Tetrachloroethylene	mg/L	0.005	11	0	-	-	-	3	0	-	-	-
Toluene	mg/L	1	11	0	-	-	-	3	0	-	-	-
Trichloroethylene	mg/L	0.005	11	0	-	-	-	3	0	-	-	-
Vinyl Chloride	m	0.002	11	0	-	-	-	3	0	-	-	-
Xylenes (total)	m	10	11	0	-	-	-	3	0	-	-	-
Radionuclides												
Gross Alpha, Excl. Radon & U	pCi/L	15	3	0	-	-	-	1	1	-	-	2.4
Gross Beta Particle Activity	pCi/L	50	2	2	2.7	3.7	3.2	-	-	-	-	-
Radium-226 and Radium-228	pCi/L	5	3	0	-	-	-	1	0	-	-	-
Radon	pCi/L	300	-	-	-	-	-	-	-	-	-	-
Combined Uranium	mg/L	0.03	3	0	-	-	-	1	0	-	-	-
Disinfectant Residuals and Disinfection By-products (DBPs)												
Chlorine	mg/L	4	0.4 - 1.7	0.87	0.44	0.85	1.3	0.2 - 2.0	0.64	0.3	0.63	1

Note:
(1) $CaCO_3$: calcium carbonate DBP: Disinfection By-product MCL: maximum contaminant level MFL: million fibers per liter pCi/L: picoCuries per liter.

5.3.2 Regulatory Overview

This section discusses current and future potential water quality regulations of interest to the City's treatment system.

5.3.2.1 Oregon Drinking Water Quality Act

The Oregon Drinking Water Quality Act, which includes the Oregon Revised Statutes (ORS), was enacted in 1981 and has been periodically amended since. According to the OHA), the Act has three purposes:

- Ensure safe drinking water for all Oregonians.
- Provide a simple and effective regulatory program for drinking water systems.
- Improve inadequate drinking water systems.

ORS 448.131 authorizes the OHA to adopt administrative rules that ensure safe drinking water. Oregon Administrative Rule (OAR) Chapter 333 Division 061 is reserved for public water system regulations.

5.3.3 Existing Regulations

The City must comply with the following state and federal drinking water regulations:

- National Primary Drinking Water Regulations (1975).
- Secondary Drinking Water Regulations (1979, 1991).
- Phases I, II, and V Regulations for IOCs, SOCs, and VOCs (1987, 1991, 1992; respectively).
- Surface Water Treatment Rule (1989).
- Consumer Confidence Reports Rule (1998).
- Stage 1 & Stage 2 Disinfectant/Disinfection By-Product Rule (D/DBPR) (2006).
- Total Coliform Rule (1989).
- Groundwater Rule (2006).
- Lead and Copper Rule (1991).
- Long-Term Stage 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) (2006).

These regulations were either proposed or implemented under the Safe Drinking Water Act (SDWA), which safeguards public health by regulating the nation's public drinking water supplies and protecting drinking water and its sources. According to the 1996 amendments to the SDWA, each state must develop a Source Water Assessment and Protection (SWAP) Program outlining how it will assess its public water supplies.

The following subsections summarize the City's compliance with the most recent water quality regulations.

5.3.3.1 Disinfection By-Products

DBPs form in the distribution system when free chlorine reacts with naturally occurring organic substances in water. Some disinfectants and DBPs have shown to cause cancer and birth defects in lab animals and are suspected to cause bladder cancer and birth defects in humans. The D/DBPR regulates DBPs by reducing the public's exposure to them from drinking water. The rule was implemented in two stages, with the second stage providing more stringent control measures.

The Stage 1 D/DBPR regulates any water system that introduces a disinfectant during any part of the treatment process. Disinfectants include Trihalomethanes (TTHMs), Haloacetic acids (HAA5), bromate, and chlorite. Under the Stage 1 D/DBPR, the MCL for TTHM is 0.08 mg/L and the MCL for HAA5 is 0.06 mg/L. Compliance is based on the running annual average (RAA) of the quarterly results. A quarterly result is the average of the results from all of the sampling locations taken that quarter.

The Stage 2 D/DBPR was implemented to strengthen the regulations for public water systems detailed in Stage 1 and was aimed at public water systems with the greatest risk of exposure. According to this rule, systems must conduct an Initial Distribution System Evaluation (IDSE) to identify compliance monitoring sites for the DBP MCLs.

The main difference between Stage 1 and Stage 2 D/DBPR is the compliance calculation of TTHM and HAA5. Stage 1 D/DBPR compliance is based on a system-wide RAA, while Stage 2 D/DBPR is based on RAA at each location, which is referred to as the locational running annual average (LRAA). Under the Stage 2 D/DBPR, the MCLs for TTHM and HAA5 remain the same as the Stage 1 D/DBPR.

Results and Recommendations

Under Oregon Public Health mandates, the City must monitor DBPs on a yearly sampling interval at two system locations. The sample points are located at 34100 Skyway Drive and 32185 Branch Road.

Table 5.3 summarizes the sampling results from the Miller Road and Keys Road WTP, based on the City's Annual Water Quality Report. As shown, DBPs were detected in the samples, but none exceeded the LRAA of 0.08 mg/L for total TTHMs or 0.06 mg/L for HAA5 in the distribution system. Therefore, no additional capital improvements are required to ensure the City remains in compliance with Stage 2 D/DBPR.

Table 5.3 Summary of Scappoose's Water Quality for Disinfection By-Products

Contaminant	Units	MCL	2014		2015		2016	
			Min	Max	Min	Max	Min	Max
Total (TTHMs)	mg/L	0.08	0.0103	0.0217	0.0175	0.0311	0.0142	0.0257
Total (HAA5)	mg/L	0.06	ND	0.0081	0.0029	0.0075	0.0015	0.0082

5.3.3.2 Total Coliform Rule

In 1989, the U.S. Environmental Protection Agency (USEPA) established the Total Coliform Rule (TCR) to reduce the risk of waterborne illness from disease-causing organisms found in animal or human waste. Coliform bacteria comprise a broad category of organisms routinely monitored in potable water supplies. Although not all coliform bacteria are pathogenic, they are relatively easy to identify in laboratory analysis. If coliform bacteria are detected, pathogenic organisms may also be present.

Bacterial contamination in a water supply can cause numerous waterborne diseases. As a result, the OHA strictly monitors and regulates these tests.

The TCR specifies two types of MCL violations: "monthly" and "acute." If a monthly or acute MCL violation occurs, a purveyor must notify both OHA and system consumers. A violation of bacteriological MCLs occurs during routine sampling when the following criteria are met:

- Coliform is detected in 5 percent or more routine or repeat samples in one month, but no follow-up violations occur (Monthly MCL).
- Coliform is present in any repeat sample collected as a follow-up to a sample with fecal coliform or *E. coli* (Acute MCL).
- Fecal coliform or *E. coli* is present in any repeat sample collected as a follow-up to a sample with coliform presence (Acute MCL).

The TCR also requires secondary disinfection in accordance with the following:

- A sample with heterotrophic plate counts (HPCs) less than 500 colony forming units per 100 milliliter (mL) is assumed to carry the required minimum residual.

The original 1989 TCR rule was revised on February 13, 2013, and was promulgated on April 1, 2016. According to this rule, public water systems vulnerable to microbial contamination must identify and fix any problems related to that contamination. The rule also established criteria that systems must meet to qualify for and remain on reduced monitoring, which could reduce burden on the water system and incentivize better system operation.

The revised rule focuses primarily on eliminating the total coliform MCL. Before the revision, positive coliform samples alone triggered corrective action or notification. With the new rule, positive coliform samples trigger only an assessment for fecal indicators, which then leads to corrective actions.

The City views this revision positively, since the public does not have to be notified when total coliform samples do not indicate a public health risk.

Results and Recommendations

Routine samples collected by Oregon public water suppliers are regularly analyzed for total coliform bacteria. The number of monthly samples required is based on the population served. For the City, at least 8 samples per month are required.

In 2018, the City has collected 8 routine samples per month and 4 assessment samples per year based on a residential population of 6,800. This information comes from the OHA's Drinking Water Data Online.

In 2016, the City had one total coliform exceedance, the same number of exceedances it had in 2008. The City completed follow-up tests on that exceedance, which were all negative. Thus, no public notification was required. The City has updated its most recent coliform test results and is currently in full compliance with the TCR. No additional capital improvements are required to ensure continued compliance with the TCR.

5.3.3.3 Groundwater Rule

In 2006, the USEPA implemented the Groundwater Rule (GWR) to increase protection against microbial pathogens and fecal-related bacteria in public groundwater systems. The GWR builds on the TCR by listing additional steps to take when a routine sample tests positive for total coliform. It also lists the sequence of actions to follow if any triggered source sample tests positive for fecal indicators, such as *E. coli*. The rule applies to all public water systems that serve groundwater, including the Miller Road and Keys Road Water Systems.

To implement the GWR, the USEPA has taken a risk-based approach that protects drinking water from groundwater sources with the greatest risk of fecal contamination. This approach is carried out using the following measures:

1. **Sanitary Surveys:** Sanitary surveys must be conducted every three years and must meet the provisions of the 1998 Interim Enhanced Surface Water Treatment Rule as it relates to the populations served. The sanitary surveys shall also implement the eight elements of the USEPA/State Joint Guidance on Sanitary Surveys. These elements govern source protection; identify the physical components and their condition; describe and govern the implementation of programs for treatment, distribution, storage, pumping, monitoring, operation, and maintenance; and govern operator certification.
2. **Source Water Monitoring:** Source water monitoring is triggered when a system does not disinfect drinking water sufficiently to achieve 4-log (99.99 percent) virus removal, and a positive routine sample is identified during its TCR monitoring and hydrogeologic sensitivity assessment monitoring (at the State's discretion). Both rules target high-risk systems. Once a total coliform-positive sample is found, the distribution system must collect one source water sample per source and monitor for a fecal indicator. Oregon may issue a waiver if the groundwater source has a hydrogeologic barrier.
3. **Corrective Action:** According USEPA guidelines, "… groundwater systems that have a significant deficiency or have detected a fecal indicator in their source water …" must take corrective actions. These actions include eliminating the contaminate source, correcting significant deficiencies, or providing an alternate source of water supply, which must occur within 90 days of the detection. This timeframe could, however, be extended with State approval.
4. **Compliance Monitoring:** Compliance monitoring ensures that treatment technology installed to treat drinking water reliably achieves 4-log virus inactivation and applies to all groundwater systems that disinfect as a corrective action. Systems that serve 3,300 individuals or more must continuously monitor their disinfection treatment process. If disinfection concentrations are below the required level, the system must restore disinfection concentration within four hours.

Source water monitoring is required at all sources where a distribution system sample tests positive for total coliform (as collected under the total coliform regulations). However, the federal GWR has a provision where positive coliform samples attributed to a distribution system source do not trigger source water monitoring if the coliform positive sample was attributed to a deficiency in the distribution system. OHA has not yet decided on the criteria for this.

Results and Recommendations

Source water monitoring will be required at fewer sources if systems can demonstrate which sources affect each TCR sample site. However, such a plan would need OHA preapproval. The federal GWR also allowed for reduced source water monitoring after 12 non-detect samples. However, OHA has yet to establish a standard for this as well.

The City completed its sanitary survey in 2016 and had one coliform exceedance. Follow-up tests showed negative results. Thus, no public notification and further action was required. The City is currently in full compliance with the GWR; no additional capital improvements are required to ensure continued compliance. The City's sanitary survey can be found in Appendix A.

5.3.3.4 Lead and Copper Rule

Lead and copper are heavy metals sometimes found in household plumbing materials and water service lines. Promulgated in 1995, the Lead and Copper Rule (LCR) was established to reduce tap water concentrations of lead and copper that can occur when corrosive source water causes these metals to leach from water meters and plumbing fixtures. The LCR primarily addresses the effects of corrosive water on older plumbing installed between 1982 and 1986, before lead solder was banned from use on plumbing fixtures.

No lead and copper MCLs have been established yet. Instead, the LCR establishes an action level (AL) of 0.015 mg/L for lead and 1.3 mg/L for copper, based on the 90th percentile level of tap water sample test results. An AL exceedance is not a violation. However, it can trigger requirements to protect customer health, such as water quality parameter monitoring, corrosion control treatment, source water treatment, public education, and lead service line replacement. Per Oregon Public Health, the City must monitor for lead and copper every three years.

Results and Recommendations

Under the LCR, lead and copper samples are collected from customers' cold water taps at homes and buildings at high risk of lead or copper contamination. The City is currently on a reduced monitoring schedule for lead and copper and is required to collect lead and copper samples at 20 taps in its distribution system once every three years. Table 5.4 summarizes the sampling results from the Miller Road and Keys Road WTP. As shown, the 90th percentile results did not exceed the action level. Thus, the City does not need to carry out additional actions and is in full compliance with the LCR.

Table 5.4 Scappoose's Lead and Copper 90th Percentile Summary Results

Contaminants	Units	AL	2017		2014	
			Sample Count	90th Percentile	Sample Count	90th Percentile
Lead	mg/L	0.015	21	0.0	20	0.0
Copper	mg/L	1.3	21	0.137	20	0.123

5.3.3.5 Long Term 2 Enhanced Surface Water Treatment Rule

The LT2ESWTR was finalized on January 5, 2006. Under the LT2ESWTR, systems must monitor their water sources to determine if additional treatment is required to remove *Cryptosporidium.* This monitoring includes an initial two years of monthly sampling for *Cryptosporidium* and *E. coli*. Filtered water systems are classified in one of four treatment categories (bins) based on *Cryptosporidium* monitoring results.

Results and Recommendations

The City initiated the two year monitoring program in October 2017 and is currently monitoring *E. coli* samples every two weeks. It is recommended that the City continue with the monitoring program for LT2ESWTR. No additional capital improvements are anticipated to ensure continued compliance with the LT2ESWTR.

5.3.4 Future Regulations

The following regulatory actions are anticipated for the near future:

- Final Fourth Unregulated Contaminant Monitoring Rule (UCMR 4).
- Regulatory changes resulting from National Drinking Water Advisory Council's (NDWAC) Lead and Copper Rule Working Group's final report and recommendations.

These regulatory actions are not expected to affect the treatment process recommendations. Nonetheless, they are covered in the recommended raw water sampling plan and emerging contaminants of concern sections below.

5.3.4.1 Unregulated Contaminant Monitoring

The USEPA manages the Unregulated Contaminant Monitoring (UCM) program directly, using it to collect data on contaminants that are suspected in drinking water but have no health-based standards under the SDWA. The UCM program has been altered and updated several times throughout its history. Milestones in its development are described below:

- UCM – State Rounds 1 and 2 (1988-1997) – State drinking water programs managed the original program, which required public water systems (PWSs) serving more than 500 people to monitor contaminants.
- UCMR 1 (2001-2005) – The SDWA Amendments of 1996 redesigned the UCM program to incorporate a tiered monitoring approach and required monitoring for 25 contaminants (24 chemicals and one bacterial genus) between 2001 and 2003.
- UCMR 2 (2007-2011) – The USEPA managed UCMR 2 monitoring, which established a new set of 25 chemical contaminants sampled between 2008 and 2010.
- UCMR 3 (2012-2016) – UCMR 3 monitoring was completed in March 2013. Table 5.5 lists the contaminants sampled and the range of contaminants detected in the Northwest to provide the City perspective on its source water quality. Briefly, a limited number of the UCMR 3 contaminants were found in the City's source water, however, when detected, these contaminants occurred at the low end of the detected range for region.
- Sampling for the UCMR 4 will begin July 2019. Table 5.6 provides a list of the proposed compounds to be analyzed.

Table 5.5 Summary of Scappoose and Other Regional WTPs UCMR 3 Finished and Distribution Water Quality

Contaminant	Range of Detects OR/WA (µg/L)(1)	Range of Detects Wilsonville (µg/L)	Range of Detects Corvallis (µg/L)	Range of Detects McKenzie (µg/L)	Range of Detects Salem (µg/L)	Scappoose	
						Range of Detects in 2013-2014 (µg/L)	Average Detects (µg/L)
1,1-dichloroethane	0.036	-	-	-	-	-	-
1,2,3-trichloropropane	-	-	-	-	-	-	-
1,3-butadiene	-	-	-	-	-	-	-
1,4-dioxane	0.07 – 0.28	-	-	-	-	-	-
17-alpha-ethynylestradiol	-	-	-	-	-	-	-
17-beta-estradiol	-	-	-	-	-	-	-
4-androstene-3,17-dione	0.0004	-	-	-	-	-	-
bromomethane	-	-	-	-	-	-	-
chlorate	20 - 3000	43 – 130	160 – 330	-	82 – 100	39.7 – 166.16	78.09
chloromethane	0.2 – 2.2	-	-	-	-	-	-
chromium	0.2 – 55	0.2	0.34 – 0.48	0.39	0.26	-	-
Chromium-6	0.03 – 4.0	0.038 – 0.072	0.093 – 0.32	0.098 – 0.12	0.042 – 0.049	0.031 – 0.05	0.038
cobalt	1.8 – 1.9	-	-	-	-	-	-
Equilin	-	-	-	-	-	-	-
Estriol	-	-	-	-	-	-	-
Estrone	-	-	-	-	-	-	-
Halon 1011	0.087 – 1.0	-	-	-	-	-	-
HCFC-22	0.088 – 0.67	-	-	-	-	-	-
Manganese	1 – 820	-	-	-	-	1.152 – 2.79	1.85
Molybdenum	1 – 13	-	-	-	-	-	-
PFBS	-	-	-	-	-	-	-
PFHpA	0.013 – 0.026	-	-	-	-	-	-
PFHxS	0.20 – 0.24	-	-	-	-	-	-
PFNA	0.027 – 0.028	-	-	-	-	-	-

Table 5.5 Summary of City and Other Regional WTPs UCMR 3 Finished and Distribution Water Quality (Continued)

Contaminant	Range of Detects OR/WA (µg/L)(1)	Range of Detects Wilsonville (µg/L)	Range of Detects Corvallis (µg/L)	Range of Detects McKenzie (µg/L)	Range of Detects Salem (µg/L)	Scappoose	
						Range of Detects in 2013-2014 (µg/L)	Average Detects (µg/L)
PFOA	0.02 – 0.03	-	-	-	-	-	-
PFOS	0.51 – 0.60	-	-	-	-	-	-
Strontium	0.9 – 531	36 – 41	29 – 40	25 – 28	20 – 24	37.3 – 88.24	55.23
Testosterone	0.0005	-	-	-	-	-	-
Vanadium	0.2 – 41.9	1.0 – 2.5	1.8 – 3	3.9 – 5.6	0.98 – 1.7	0.3 – 2.73	1.62

Note:
(1) µg/L: micrograms per liter.

Table 5.6 Proposed UCMR4 Sampling List, City of Scappoose

Contaminant	Type	Health Reference Level	Minimum Reporting Level	Critical Health Effect
total microcystin	Cyanotoxin		0.3 µg/L	
microcystin-LA		0.3 µg/L	0.008 µg/L	Liver effects
microcystin-LF		0.3 µg/L	0.006 µg/L	Liver effects
microcystin-LR		0.3 µg/L	0.02 µg/L	Liver effects
microcystin-LY		0.3 µg/L	0.009 µg/L	Liver effects
microcystin-RR		0.3 µg/L	0.006 µg/L	Liver effects
microcystin-YR		0.3 µg/L	0.02 µg/L	Liver effects
Nodularin		NA	0.005 µg/L	Liver Toxicity
anatoxin-a		NA	0.03 µg/L	Nervous System
cylindrospermopsin		0.7 µg/L	0.09 µg/L	Increased relative kidney weight & decreased urinary protein
Germanium	Metal	7.44 µg/L	0.3 µg/L	Kidney, ureter, bladder changes in tubules
Manganese		300 µg/L	0.4 µg/L	Central nervous system effects

Table 5.6 Proposed UCMR4 Sampling List, City of Scappoose (Continued)

Contaminant	Type	Health Reference Level	Minimum Reporting Level	Critical Health Effect
1-butanol	Alcohols	700 µg/L	2.0 µg/L	Abnormally diminished activity in the body/organs; inability to control muscles
2-methoxyethanol		21 µg/L	0.4 µg/L	Reproductive effects
2-propen-1-ol		35 µg/L	0.5 µg/L	Impaired kidney function and increased relative liver, spleen and kidney weights
HAA5	Brominated Haloacetic Acid (HAA) Groups			
HAA6Br				
HAA9				
alpha-hexachlorocyclohexane	Pesticides/ Pesticide Manufacturing Byproduct	0.006 µg/L	0.01 µg/L	Cancer
Chlorpyrifos		NA	0.03 µg/L	Significant plasma and RBC cholinesterase inhibition
Dimethipin		153 µg/L	0.2 µg/L	Kidney, lungs, duodenum, liver, glandular stomach, heart, aortic artery, and testes toxicity; decreased body weight gain
Ethoprop		1.25 µg/L	0.03 µg/L	Cancer
Oxyfluorfen		210 µg/L	0.05 µg/L	Liver toxicity
Profenofos		0.35 µg/L	0.3 µg/L	Plasma and RBC cholinesterase inhibition
tebuconazole		210 µg/L	0.2 µg/L	Decreased body weights, absolute brain weights, brain measurements and motor activity in offspring
Total permethrin (cis- & trans-)		3.65 µg/L	0.04 µg/L	Cancer
Tribufos		7 µg/L	0.07 µg/L	Plasma cholinesterase (ChE) inhibition
butylated hydroxyanisole	Semi volatile Chemicals	0.581 µg/L	0.03 µg/L	Changes in liver weight
o-toluidine		0.194 µg/L	0.007 µg/L	Cancer
Quinoline		0.01 µg/L	0.02 µg/L	Cancer

5.3.4.2 Lead and Copper Rule Revisions

The USEPA is proposing revisions to identify additional actions that will equitably reduce the public's exposure to lead and copper when corrosion control treatment alone is not effective. These revisions may require all public utilities to review and update their sampling site locations and protocols.

The USEPA's revisions will include requirements for optimal corrosion control treatment. As a result, the City should wait to evaluate its corrosion control treatment needs after the regulatory requirements are published. However, no capital improvements to ensure continued compliance are currently anticipated.

5.3.5 Summary

Finished supply water quality meets and/or exceeds all applicable current and future anticipated regulatory requirements. Therefore, no regulatory Capital Improvement Plans (CIP's) are recommended to ensure continued compliance with future regulations.

However, the City should continue to closely monitor several water quality parameters, and the fate of future regulation of these parameters, as these parameters may require future improvements, including:

- Revisions to the LCR may require changes to the sampling location and protocols. The City should reevaluate corrosion control treatment requirements once proposed revisions are published.
- The USEPA has found sufficient evidence of health impacts (neurotoxicity) from Manganese to support regulations. UCMR4 requires all utilities to sample for Manganese and will make a regulatory determination in the future. Beginning regular sampling for Manganese in raw and treated groundwater and surface water is recommended to better anticipate potential implications of these pending rules on the City's treatment infrastructure.
- Algal toxins are a growing concern for Oregon utilities using surface water sources. UCMR4 requires sampling for algal toxins. Thus, Carollo recommends continuing watershed best management practices that limit stream conditions conducive to algal growth. If algal toxins are found in the future, additional treatment of surface water supplies may be required.

5.4 Treatment Plant Evaluation

5.4.1 Evaluation Criteria

On November 8, Carollo completed a site tour, led by the City, to evaluate the Miller Road and Keys Road Treatment Plants. The site tour was conducted to identify and observe the major components of each facility, document the existing condition, and discuss any ongoing issues with operations staff.

After the tour, each major facility component was scored on four categories: condition, capacity, plant performance optimization, and reliability/redundancy. Positive, neutral, or negative ratings were assigned for the evaluation. The detailed evaluation criteria are shown in Table 5.7.

On February 1, a follow-up meeting was conducted with plant staff to present the preliminary evaluation based on the November site tour. This meeting was conducted to review existing evaluations and identify any other issues or concerns plant staff had experienced.

Table 5.7 Facility Evaluation Criteria

Assessment Category	Positive	Neutral	Negative
Seismic/Life-Safety Condition	Structurally sound; no noted/historical failure in mechanical equipment; facility is in great condition relative to its age.	Structural condition is moderate; mechanical components serviceable, but they consistently require maintenance; facility is in moderate condition relative to its age; no signs of failure in structures or mechanical equipment.	Visible structural or immanent mechanical failure; facility is in poor condition relative to its age.
Capacity	Has historically met demands; can meet true design capacity.	Has historically met demands; likely unable to meet true design capacity/future plant demands.	Limited capacity that has historically not met demands or is unable to meet true design capacity/future demands.
Plant Performance Optimization	Has historically met performance goals.	Has historically met performance goals with increased operational/ maintenance attention.	Has had significant historical performance issues; unable to meet performance goals.
Reliability/ Redundancy	Has sufficient redundancy/firm capacity; both structural and mechanical elements are proven reliable; likely able to handle anticipated future emergency/failure events.	Has limited redundancy; structures or mechanical equipment have not proven to always be reliable, but likelihood of emergency/failure is moderate.	Has limited or no redundancy; structures or mechanical parts have history of failure; low likelihood of handling a future emergency/failure event.

The following sections summarize the overall performance and treatment system conditions of these facilities.

5.4.2 Miller Road WTP

Table 5.8 summarizes the capacity of each existing treatment process at Miller Road WTP. This section also provides a detailed summary of the plant facilities.

Table 5.8 Miller Road Water Treatment Plant Existing Facilities Design Criteria

Description	Units(1)	Value
Miller Road Wells		
Number of Wells	#	3
Capacity, each	mgd	0.2 – 0.3
Number of Pumps	#	3
Pumping Capacity, MR-1	gpm	450
Pumping Capacity, MR-2	gpm	450
Pumping Capacity, MR-3	gpm	450
Depth	ft	190
Well Casing		
Diameter	inches	8
Liner	inches	6
Well Screen		
Chemical Systems		
Potassium Permanganate		
Number of Tanks	#	1
Nominal Size of Tanks	gal	260
Feed Pumps		
Number of pumps	#	1
Pumping Capacity, each	gph	24
Dosage Range	mg/L	
Chlorine		
Number of Hypochlorite Tanks	#	1
Nominal Size of Hypochlorite Tanks	gal	360
Number of Brine Tanks	#	1
Nominal Size of Brine Tanks	gal	275
Feed Pumps		
Number of pumps	#	2
Pumping Capacity, each	gph	Unknown
Dosage Range	mg/L	
Soda Ash		
Number of Storage Silo	#	1
Nominal Size of Storage Silo	gal	800
Feed Pumps		
Number of pumps	#	1
Pumping Capacity, each	gph	26
Dosage Range	mg/L	

Table 5.8 Miller Road Water Treatment Plant Existing Facilities Treatment Processes and Procedures (Continued)

Description	Units	Value
Fluoride		
Number of Tanks	#	1
Nominal Size of Tanks	gal	100
Feed Pumps		
Number of Pumps	#	1
Pumping Capacity, each	gph	4.5
Dosage Average	mg/L	
Filters		
Type: Greensand		
Number of Filters	#	2
Filter Backwash Water	gal	26,000
Filter to Waste	gal	3,250
Effluent Pumps		
Number of Pumps	#	2
Pumping Capacity, each	gpm	650
Filter No. 1		
Filter Area	sf	216
Capacity, Design	mgd	0.93
Filter No. 2		
Filter Area	sf	216
Capacity, Design	mgd	0.93
Backwash Basin		
Capacity, Design	mgd	0.126
Total Volume	cf	16,810
Depth	ft	5
Filter Backwash Water	gpd	104,000
Filter to Waste	gpd	13,000
Booster Pump Station		
Number of pumps	#	2
Capacity, each	mgd	0.936
Total Dynamic Head	ft	205
Power Supply		
Number of Generators	#	1
Power, Generator 1	kW	350

Note:
(1) cf: cubic feet ft: feet gal: gallon gpd: gallons per day gph: gallons per hour gpm: gallons per minute kW: kilowatt mgd: million gallons per day sf: square feet.

5.4.2.1 Groundwater Wells

Three active groundwater wells on-site, labeled MR-1, MR-2, and MR-3, supply water to Miller Road. These wells were originally constructed in 2001, 2003, and 2003, respectively, and were rehabilitated in 2015. Their total production capacity is 0.3 mgd.

Condition: The groundwater wells received a positive condition rating. Although signs of minor leakage were noted from the mechanical equipment, the components were in good condition relative to the facility's age.

Capacity: The groundwater wells received a positive capacity rating. Their total production capacity continues to meet current demands. However, production has declined since start-up.

Plant Performance Optimization: The groundwater wells received a positive performance rating. The plant runs two wells at a time due to decreased production, which staff has not reported any performance issues with.

Reliability/Redundancy: The groundwater wells received a positive reliability/redundancy rating. The multiple wells on-site provide sufficient reliability and redundancy and can pump directly into the distribution system.

5.4.2.2 Chemical Systems

Built in 2004, the chemical system consists of pretreatment permanganate, pretreatment chlorine, post-treatment soda ash, and post-treatment fluoride. All chemical systems contain a single storage tank and a single feed pump, except for chlorine.

Condition: The chemical system received a neutral condition rating. The equipment is well maintained and appeared in moderate condition relative to the facility's age.

Capacity: The chemical system received a positive capacity rating. The system continues to meet demands when required.

Plant Performance Optimization: The chemical system received a positive performance rating. This system has no known performance issues. Operators have reported chemical siphoning with permanganate and dry chemical hardening in the fluoride/soda ash feed lines. However, these issues have been addressed.

Reliability/Redundancy: The chemical system received a neutral reliability/redundancy rating. Chemical feed lines have only one feed pump, which does not provide sufficient redundancy.

5.4.2.3 Filters

Greensand Filter No. 1 and 2 were built in 2004, each with a capacity of 0.93 mgd. Due to leaks and cracks in the concrete, Filter No. 1 was rehabilitated in 2012. Under normal operations, the filters are run in a 1+1 configuration due to the declining production of the groundwater wells.

Condition: Filter No. 1 and 2 received a positive condition rating. No cracks were observed in the concrete, and none of the equipment failed during the walkthrough. The facility was in good condition given its age.

Capacity: Filter No. 1 & 2 received a positive capacity rating. The filters continue to meet demands when required.

Plant Performance Optimization: Filter No. 1 & 2 received a positive performance rating. The filters do not have any known performance issues.

Reliability/Redundancy: Filter No. 1 & 2 is positive. The filters are in a 1+1 configuration, which provides sufficient redundancy.

5.4.2.4 Backwash Basin

The Backwash Basin was built in 2004 with an initial capacity of 0.126 mgd. Shortly after construction, plant staff experienced the basin lifting up from the ground. To resolve this issue, eco-blocks were placed to weigh down the basin. The capacity of the basin was significantly reduced due to the volume of the blocks.

Condition: The Backwash Basin received a neutral condition rating. No cracks were seen in the concrete, and none of the mechanical equipment failed during the walkthrough. In general, the facility appeared to be in moderate condition relative to its age.

Capacity: The Backwash Basin received a positive capacity rating. The facility has historically met demands when required, but improvements may have compromised the capacity.

Plant Performance Optimization: The Backwash Basin received a positive performance rating. It has no known performance issues.

Reliability/Redundancy: The Backwash Basin received a neutral reliability/redundancy rating. With only one basin, there is insufficient redundancy.

5.4.2.5 Booster Pump Station

The BPS was built in 2004 and consists of two Paco vertical split-case pumps. Each pump's capacity is limited to 0.936 mgd.

Condition: The BPS received a positive condition rating. The pumps were in good condition relative to the facility's age. However, plant staff has observed moisture on one of the pump's lower bearings.

Capacity: The BPS received a positive capacity rating. The pumps continue to meet demands when required.

Plant Performance Optimization: The BPS received a positive performance rating. The pumps have no known performance issues.

Reliability/Redundancy: The BPS received a neutral reliability/redundancy rating. The pumps are operated in a 2+0 configuration because the pump alignment is mirrored. This configuration does not provide sufficient redundancy.

5.4.2.6 Power Supply

Power is supplied to the Miller Road WTP site by a 350 kW primary generator and standby generators located by the administration and operations building.

Condition: The power supply received a positive condition rating. This system is in good condition relative to the facility's age.

Capacity: The power supply received a positive capacity rating. This system continues to meet demands.

Plant Performance Optimization: The power supply received a neutral performance rating. Plant staff has reported issues with glitches in the system. Overall, the system meets its performance goals, but increased operational/maintenance attention is required.

Reliability/Redundancy: The power supply received a positive reliability/redundancy rating. The primary overhead power supply has a backup to the primary feed, which provides strong redundancy.

5.4.2.7 Laboratory

Water quality testing and monitoring for the Miller Road WTP occur in the laboratory, which is located in the administration and operations building.

Condition: The laboratory received a positive condition rating. No cracks were seen in the concrete, and all equipment appeared to be in good condition relative to the facility's age.

Plant Performance Optimization: The laboratory received a positive performance rating. However, it is recommended that the City include an oxidation/reduction potentiometer (ORP) to help detect/alarm permanganate overdosing in the finished water.

Reliability/Redundancy: The laboratory received a positive reliability/redundancy rating. The facility is well equipped with necessary devices and safety measures, which provides sufficient redundancy.

5.4.2.8 Miller Road WTP Assessment Summary

Each of the above components was assigned a positive, neutral, or negative rating for its condition, capacity, plant performance optimization, and reliability/redundancy. Table 5.9 summarizes these ratings.

Table 5.9 Miller Road Water Treatment Plant Assessment Summary

Facility	Condition	Capacity	Plant Performance Optimization	Reliability/ Redundancy
Groundwater Wells				
Chemical Systems				
Filters No. 1 & 2				
Backwash Basin				
Booster Pump Station				
Power Supply				
Laboratory		N/A		

5.4.2.9 Recommended Improvement Alternatives

Based on the preliminary facility assessment evaluation, improvement alternatives were recommended for facilities that received a yellow or red rating. Table 5.10 and summarize these alternatives, which are also incorporated into Table 5.11, the recommended improvement alternative CIP.

Table 5.10 Miller Road Water Treatment Plant Recommended Improvement Alternatives Summary

Item	Recommendation
Chemical Systems	• Incorporate spill containment to address safety challenges for plant staff. • Identify and procure critical spare parts to improve redundancy/reliability.
Backwash Basin	• Build a new basin to accommodate increased capacity and improve redundancy/reliability.
Booster Pump Station	• Add additional shelf spare. • Identify and procure critical spare parts to improve redundancy/reliability.
Power Supply	• Refine SCADA(1) as needed to improve performance.
Laboratory	• Consider testing both raw and finished water manganese; • Consider adding an ORP sensor to the plant effluent to monitor permanganate.

Note:
(1) SCADA: Supervisory Control and Data Acquisition.

5.4.2.10 CIP Summary

Table 5.11 presents the CIP summary table for the recommended improvement alternatives. These CIP projects are necessary to repair and maintain existing system facilities and to meet the needs of projected growth. For up to date timing of these CIP projects see Chapter 8 - Capital Improvement Plan.

The cost estimate presented is an American Association of Cost Engineers (AACE) Class 5 estimate. This is a concept screening level estimate with approximately 2 percent of the design defined, with an expected accuracy range of +100 percent to -50 percent. It is subject to change in the future.

Table 5.11 Miller Road WTP Recommended Improvement Alternatives CIP

Project	Estimated Cost	Note
Repair and Replacement	**$654,000.00**	
Chemical Systems	$65,000.00	
Backwash Basin	$500,000.00	
Booster Pump Station	$50,000.00	
Power Supply	$30,000.00	
Laboratory	$9,000.00	TBD with City

Note:
(1) TBD: To Be Determined.

5.4.3 Keys Road WTP

Table 5.12 summarizes the capacity of each existing treatment process at Keys Road WTP. This section also includes a detailed summary of the plant facilities.

Table 5.12 Keys Road Water Treatment Plant Existing Facilities Design Criteria

Description	Units(1)	Value
Plant Design Flow		
Design Flow, Average	mgd	1.15
Design Flow, Maximum	mgd	2.88
Surface Water Intake		
Capacity, Winter	mgd	2
Capacity, Summer	mgd	0.36
Groundwater Wells		
Number of Wells	#	2
Capacity, each	mgd	0.5+
Depth	ft	228
Well Casing		
Diameter	inches	23
Well Screen		
Diameter	inches	10
Number of Pumps	#	+1
Pumping Capacity	gpm	400
Total Dynamic Head	ft	460
Chemical Systems		
Pre-treatment Chlorine		
Number of Tanks	#	1
Nominal Size of Tanks	gal	540
Feed Pumps		
Number of pumps	#	1
Pumping Capacity, each	gph	4.5
Dosing Average	mg/L	3.0
Feed Rate, Average Flow-Average Dose	lb/day	30
Feed Rate, Design Flow-Max Dose	lb/day	240
Pre-treatment Aluminum Sulfate		
Number of Storage Tanks	#	1
Nominal Size of Storage Tanks	gal	6,000
Diameter	ft	12
Feed Pumps		
Number of pumps	#	1
Pumping Capacity, each	gph	24
Dosing Average	mg/L	30
Feed Rate, Average Flow-Average Dose	lb/day	290
Feed Rate, Design Flow-Max Dose	lb/day	2400

Table 5.12 Keys Road Water Treatment Plant Existing Facilities Design Criteria (Continued)

Description	Units(1)	Value
Pre-treatment Polymer		
Coagulation		
Dosing Average	mg/L	0.1
Feed Rate, Average Flow-Average Dose	lb/day	1.0
Feed Rate, Design Flow-Max Dose	lb/day	25
Filter Aid		
Dosing Average	mg/L	0.005
Feed Rate, Average Flow-Average Dose	lb/day	0.05
Feed Rate, Design Flow-Max Dose	lb/day	0.24
Sludge Conditioner		
Dosing Average	mg/L	1.0
Feed Rate, Average Flow-Average Dose	lb/day	0.08
Feed Rate, Design Flow-Max Dose	lb/day	0.29
Pre-treatment Permanganate		
Number of Tanks	#	
Nominal Size of Tanks	gal	300
Diameter	ft	
Feed Pumps		
Number of pumps	#	1
Pumping Capacity, each	gph	14.3
Dosing Average	mg/L	0.5
Feed Rate, Average Flow-Average Dose	lb/day	5
Feed Rate, Design Flow-Max Dose	lb/day	120
Post-treatment Caustic Soda		
Number of Storage Tanks	#	1
Nominal Size of Storage Tanks	gal	6,000
Diameter	ft	12
Feed Pumps		
Number of Pumps	#	1
Pumping Capacity, each	gph	4.8
Dosing Average	mg/L	10
Feed Rate, Average Flow-Average Dose	lb/day	95
Feed Rate, Design Flow-Max Dose	lb/day	720
Post-treatment Fluoride		
Number of Storage Tanks	#	1
Nominal Size of Storage Tanks	gal	50
Diameter	ft	

Table 5.12 Keys Road Water Treatment Plant Existing Facilities Design Criteria (Continued)

Description	Units(1)	Value
Post-treatment Fluoride (continued)		
Feed Pumps		
Number of Pumps	#	1
Pumping Capacity, each	gph	4.5
Dosage Range	mg/L	
Direct Filtration Plant		
Type: Conventional		
Rapid Mix		
Type: In-line		
Size	inches	12
Flocculation		
Number of Basins	#	2
Depth	ft	6.5
Volume, each	gal	8,000
Detention Time at Average Flow	minutes	20
Tube Settler		
Design Flow	mgd	1.15
Max Overflow Rate	gpd/sf	200
Face Area Loading	gpm/sf	2.5
Filters		
Filter No. 1		
Type: Dual Media (anthracite coal, garnet sand, silica sand, and gravel)		
Number of Filters	#	2
Depth	inches	30
Total Filter Area	sf	400
Filtration Rate at Average Flow	gpm/sf	2.0
Maximum Filtration Rate	gpm/sf	5.0
Maximum Backwash Rate	gpm/sf	18.7
Surface Wash Rate	gpm/sf	1.0
Filter No. 2		
Type: Greensand		
Number of Filters	#	1
Filter Area	sf	200
Capacity, Actual	mgd	0.461
Filtration Rate, Actual	gpm/sf	2.65
Filtration Rate, Design	gpm/sf	4

Table 5.12 Keys Road Water Treatment Plant Existing Facilities Design Criteria (Continued)

Description	Units[1]	Value
Filter Wash System		
Type: Surface Wash		
Surface Wash Rate	gpm/sf	1
Backwash Pumps		
Number of Pumps	#	1
Pumping Capacity	gpm	950
Booster Pump Station		
Pump #1	#	3
Pumping Capacity	gpm	111
Total Dynamic Head	ft	268.7
Pump #2		
Pumping Capacity	gpm	90
Total Dynamic Head	ft	255
Pump #3 (Fire)		
Pumping Capacity	gpm	500
Total Dynamic Head	ft	-
Backwash Pumps		
Number of pumps	#	2
Pumping Capacity, each	gpm	3800
Total Dynamic Head	ft	45
Water Reservoirs		
Number of Reservoirs	#	5
Capacity, total	MG	3.87
Reservoir No. 1		
Type: Concrete		
Volume, Design	MG	0.2
Reservoir No. 2		
Type: Concrete		
Volume, Design	MG	1
Reservoir No. 3		
Type: Concrete		
Volume, Design	MG	2
Reservoir No. 4		
Type: Steel		
Volume, Design	MG	0.3
Reservoir No. 5		
Type: Steel		
Volume, Design	MG	0.37

Table 5.12 Keys Road Water Treatment Plant Existing Facilities Design Criteria (Continued)

Description	Units(1)	Value
Washwater Reclamation Basin		
Effective Depth	ft	10
Capacity	cf	15,000
Power Supply		
Number of Generators	#	1
Power, Generator 1	kW	250

Note:
(1) MG: million gallons gpd/sf: gallons per day per square foot gpm/sf: gallons per minute per square foot lb/day: pounds per day.

5.4.3.1 Surface Water Intake

The Surface Water Intake's sources are located at South Scappoose Creek, Gourlay Creek, and Lazy Creek. The intakes were built in 1921, 1955, and 1967, respectively. All intakes are combined and then fed into the Keys Road Treatment Plant. The winter capacity (pipeline) is limited to 2 mgd, and the summer capacity (average usage) is 0.36 mgd.

Condition: The Surface Water Intake received a neutral condition rating. No cracks were seen in the concrete; however, the system is not likely seismically resilient. Repair and replacement will be difficult because of the intake's location and permit requirements. Overall, the intakes are in moderate condition relative to the facility's age.

Capacity: The Surface Water Intake's capacity rating is unknown. Carollo recommends monitoring the flow of the intakes to determine if the system has historically met demands when required.

Plant Performance Optimization: The Surface Water Intake received a neutral performance rating. The system has historically met performance goals; however, frequent maintenance is required to remove sediment deposits. The City reported that its water availability does not align with demands.

Reliability/Redundancy: The Surface Water Intake has a positive reliability/redundancy rating. The multiple intakes in place provide sufficient redundancy.

5.4.3.2 Surface Water Transmission

The old surface water transmission mains were built in 1955 and 1968, and a new transmission main was built along SW Em Watts Road. In 2016, damage to the South Fork Diversion Dam caused damaged to the South Fork Pipeline.

Condition: The surface water transmission main's condition rating is unknown. The transmission is over 50 years old and will need to be replaced or upsized when it reaches the end of its usable life.

Capacity: The surface water transmission mains received a negative capacity rating. The system has limited capacity that has not met demands when required.

Plant Performance Optimization: The surface water transmission mains received a negative performance rating. Events causing high turbidity or sediment deposits have made the transmission mains function poorly. Overall, the system has had significant performance issues and cannot meet performance goals.

Reliability/Redundancy: The surface water transmission mains received a negative reliability/redundancy rating. In the pipeline, the transmission is a single point of failure, which does not provide sufficient reliability. Repair and replacement will be difficult because of the transmission's location and permit requirements.

5.4.3.3 Groundwater Wells

One active groundwater well south of Dutch Canyon Road supplies water to the Keys Road Water System. This well, known as Dutch Canyon Well, was originally constructed in 1978 and was rehabilitated in 2000 and 2015. Its production capacity is limited to 0.47 mgd.

The City drilled another active groundwater well in 2017. This well is expected to increase the total capacity to 0.5+ mgd.

Condition: The groundwater wells received a positive condition rating. No cracks were seen in the concrete, and the facility is in good condition relative to its age.

Capacity: The groundwater wells received a neutral capacity rating. The wells have historically met demands when required. However, they will not likely meet the true design capacity/future plant demands. The total capacity of the wells is currently lower than the total capacity of the filters.

Plant Performance Optimization: The groundwater wells received a neutral performance rating. Plant staff has reported that the original well regularly produces sand. Overall, the wells have historically met performance goals, but increased operations and maintenance are required.

Reliability/Redundancy: The groundwater wells received a positive reliability/redundancy rating. The new well provides sufficient redundancy. Overall, the system can likely handle a future emergency or failure event.

5.4.3.4 Chemical Systems

The chemical system was installed in 1979 with improvements in the early 2000s. The pretreatment processes consist of chlorine, sodium hypochlorite, aluminum sulfate, filter aid, and permanganate. Post-treatment processes consist of caustic soda, chlorine, and fluoride. All chemical systems contain a single storage tank.

Condition: The chemical systems received a negative reliability/redundancy rating. Operators experience corrosion control issues with the caustic soda line and safety challenges from a lack of containment. Overall, the chemical systems are in poor condition relative to the facility's age.

Capacity: The chemical systems received a positive capacity rating. The system continues to meet demands when required.

Plant Performance Optimization: The chemical systems received a positive performance rating. They have no known performance issues.

Reliability/Redundancy: The chemical systems received a neutral reliability/redundancy rating. The chemical lines have one feed pump, which does not provide sufficient redundancy.

5.4.3.5 Direct Filtration Plant

The Direct Filtration Plant, built in 1979, was initially designed as a conventional treatment plant. In 1993, OHA determined that it operated in direct filtration because of the plate settlers' hydraulic loading. The plant's direct filtration plant treatment processes consists of coagulation/flocculation, sedimentation, filtration, and filter to waste.

Condition: The Direct Filtration Plant received a neutral condition rating. During the walkthrough, no cracks were seen. The building and all facilities are well maintained and were in moderate condition relative to their age. However, the facility is almost 40-years-old, and the City will need to develop a long-term strategy for repair and replacement.

Capacity: The Direct Filtration Plant received a neutral capacity rating. The system has historically met demands when required. However, raw surface water is limited in the summer and fall months, which significantly reduces the capacity.

Plant Performance Optimization: The Direct Filtration Plant received a negative performance rating. Plant staff has experienced several operational challenges. Overall, the system cannot meet its performance goals.

Reliability/Redundancy: The Direct Filtration Plant received a neutral reliability/redundancy rating. Seismic improvements are likely required, and the plant has poor redundancy for a future emergency/failure event.

5.4.3.6 Greensand Filter

The greensand filter was built in 2000 with a design capacity of 0.461 mgd.

Condition: The greensand filter received a neutral condition rating. The filter is outdoors with no cover, making it vulnerable to moss and algae growth. Plant staff has reported that the filter collects leaves and fir needles. Overall, the filter is in moderate condition relative to the facility's age.

Capacity: The greensand filter received a positive capacity rating. The facility continues to meet system demands when required.

Plant Performance Optimization: The greensand filter received a neutral performance rating. Although it has historically met performance goals, increased operational/maintenance attention is required. Plant staff reported rapid headloss accumulation, which creates shorter filter runs.

Reliability/Redundancy: The greensand filter received a negative reliability/redundancy rating. Only one filter is available, offering poor redundancy and reliability during an emergency/failure event.

5.4.3.7 Booster Pump Station

The BPS, which elevates water to the higher zone's reservoirs, was built in 1979. In 2017, a pump was replaced due to catastrophic failure. The pump station consists of three pumps, each with a capacity of 111 gpm, 90 gpm, and 500 gpm.

Condition: The BPS received a positive condition rating. All of its components were in good condition relative to the facility's age.

Capacity: The BPS received a neutral capacity rating. Although it provides sufficient capacity, the future demand growth in Zone 2 is expected to exceed its firm capacity.

Plant Performance Optimization: The BPS received a neutral performance rating. The pump station has historically met performance goals. However, the fire pump over-pressurizes the system.

Reliability/Redundancy: The BPS received a positive reliability/redundancy rating. The pump station maintains three pumps, which provides sufficient redundancy and adequate reliability to meet current demands.

5.4.3.8 Backwash Pump Building

The Backwash Pump Building was built in 1979 and has two pumps, each with a capacity of 3800 gpm.

Condition: The Backwash Pump Building received a positive condition rating. All of its components were in good condition relative to the facility's age.

Capacity: The Backwash Pump Building received a positive capacity rating. The system continues to meet demands when required.

Plant Performance Optimization: The Backwash Pump Building received a positive performance rating. The backwash pumps have no known performance issues.

Reliability/Redundancy: The Backwash Pump Building received a positive reliability/redundancy rating. The pump building consists of two pumps, which provide sufficient redundancy. Staff is working to restore lead/lag operation.

5.4.3.9 Finished Water Metering and Reservoirs

Two finished water reservoirs receive water from the water plant. These reservoirs have a capacity of 0.2 MG, 1 MG, and 2MG and were built in 1946, 1967, and 2003, respectively.

Condition: The finished water metering and reservoirs received a positive condition rating. All facilities are well maintained and are in good condition relative to their age.

Capacity: The finished water metering and reservoirs received a positive capacity rating. The reservoirs continue to meet demands when required.

Plant Performance Optimization: The finished water metering and reservoirs received a neutral performance rating. Plant staff has challenges with erratic flow and metering, resulting in chlorine residual issues. To help resolve issue, we recommend installation of an air vacuum/air relief valve at the high point of the finished water pipeline to minimize impacts of entrained air in the pipeline on overall flow monitoring. Overall, the reservoirs continue to meet performance goals. However, increased operational/maintenance attention is required.

Reliability/Redundancy: The finished water metering and reservoirs received a ___(TBD)___ reliability/redundancy rating. The facilities have sufficient redundancy to meet current demands and show strong reliability with sufficient capacity. ***[NOTE: Reliability and resiliency of all the Keys Road WTP reservoirs will be addressed as part of this plan, under a separate study. This study will be included as an Appendix to this report, however, the recommended Keys Road WTP Reservoir improvements (and associated costs) from this study will be summarized in this section.]***

5.4.3.10 Power Supply

SCADA is sent to Miller Road, which is the primary operations for power. Therefore, improvements identified will be incorporated into Miller Road's recommended improvement alternatives.

Condition: The power supply received a positive condition rating. This system was in good condition relative to the facility's age.

Capacity: The power supply received a positive capacity rating. This system continues to meet demands when required.

Plant Performance Optimization: The power supply received a positive performance rating. The system has no known performance issues.

Reliability/Redundancy: The power supply received a positive reliability/redundancy rating. The primary overhead power supply has a backup to the primary feed, which provides sufficient redundancy.

5.4.3.11 Administration and Operations Building

Condition: The Administration and Operations Building received a positive condition rating. No cracks were seen in the concrete, and all equipment was in good condition.

Plant Performance Optimization: The Administration and Operations Building received a positive performance rating. It has no known performance issues.

Reliability/Redundancy: The Administration and Operations Building received a positive reliability/redundancy rating. The building has sufficient redundancy and can likely handle a future emergency/failure event.

5.4.3.12 Laboratory

Condition: The laboratory received a positive condition rating. No visible cracks were seen in the concrete, and all equipment was in good condition.

Plant Performance Optimization: The laboratory received a positive performance rating. However, it is recommended that the City include an ORP to help detect/alarm permanganate overdosing in the finished water.

Reliability/Redundancy: The laboratory received a positive reliability/redundancy rating. The facility is well equipped with the necessary devices and safety measures, which provides sufficient redundancy.

5.4.3.13 Keys Road WTP Assessment Summary

Each component discussed above was assigned a positive, neutral, or negative rating for its condition, capacity, plant performance optimization, and reliability/redundancy. Table 5.13 summarizes these ratings.

Table 5.13 Keys Road Water Treatment Plant Assessment Summary

Facility	Condition	Capacity	Plant Performance Optimization	Reliability/ Redundancy
Surface Water Intake		TBD		
Surface Water Transmission				
Groundwater Wells				
Chemical Systems				
Direct Filtration Plant				
Greensand Filter				
Booster Pump Station				
Backwash Pump Station				
Finished Water Reservoirs				TBD
Power Supply				
Admin and Operations		N/A		
Laboratory		N/A		

5.4.3.14 Recommended Improvement Alternatives

Based on the preliminary facility assessment evaluation, improvement alternatives were recommended for facilities that received a yellow or red rating. Table 5.14 and Table 5.15 summarize these alternatives, which will also be incorporated into the CIP.

Table 5.14 Keys Road Water Treatment Plant Recommended Improvement Alternatives Summary

Items	Recommendations
Surface Water Intake	• Implement seismic improvements to accommodate a future emergency/failure event. • Implement a flow monitoring program to evaluate system capacity.
Surface Water Transmission	• Replace/upsize the pipe when it reaches the end of its usable life. • Optimize the operations of preliminary sedimentation to improve system performance.
Groundwater Wells	• Continue developing the new well.
Chemical Systems	• Replace the caustic soda tank to address corrosion control issues. • Incorporate spill containment to address safety challenges for plant staff. • Identify and procure critical spare parts and provide chemical storage to improve redundancy/reliability.
Direct Filtration	• Conduct a performance optimization study. • Determine whether the plant is part of long-term strategy.
Greensand Filter	• Provide cover to improve the facility's condition. • Conduct filter optimization study to further evaluate performance.
Booster Pump Station	• Optimize pump station operations based on demand forecast. • Potentially replace fire pump to improve performance.
Finished Water Reservoirs	• Install air vacuum/air release valve at high point in pipe to resolve monitoring issues.
Laboratory	• Consider testing both raw and finished water manganese. • Consider adding an ORP sensor to the plant effluent to monitor permanganate.

5.4.3.15 CIP Summary

Table 5.15 presents the CIP summary table for the recommended improvement alternatives. These CIP projects are necessary to repair and maintain existing system facilities and to meet the needs of projected growth. For up to date timing of these CIP projects see Chapter 8 - Capital Improvement Plan.

The cost estimate presented is an AACE Class 5 estimate. This is a concept screening level estimate with approximately 2 percent of the design defined, with an expected accuracy range of +100 percent to -50 percent. It is subject to change in the future.

Table 5.15 Keys Road WTP Recommended Improvement Alternatives CIP

Project	Estimated Cost	Note
Repair and Replacement	**$342,000.00**	
Surface Water Intake	3,000.00	
Surface Water Transmission	N/A	Refer to Recommended Additional Studies
Groundwater Wells	N/A	Refer to Capacity CIP (Chapter 4)
Chemical System	$175,000.00	
Direct Filtration	N/A	Refer to Recommended Additional Studies
Greensand Filter	$100,000.00	
Booster Pump Station	$50,000.00	
Finished Water Reservoirs	$5,000.00	
Laboratory	$9,000.00	TBD with City

5.4.4 Recommended Additional Studies

In addition to the improvement alternatives shown in Table 5.15, additional studies are recommended. These studies are summarized below.

5.4.4.1 Supply and Treatment Plant LOS Goals

Carollo recommends developing Level of Service (LOS) goals for the performance of existing facilities and long-term supply alternatives that represent the City's overall water system goals. LOS goals improve communication and balance the City Council's long-term, broader expectations with the everyday operations and problem solving required of City staff.

5.4.4.2 Seismic and Life-safety Audit on all Treatment Facilities

A seismic and life-safety audit program is also recommended to identify potential seismic performance deficiencies (that can jeopardize safe, reliable operation of treatment plants) as well as potential life-safety deficiencies in the structural connections, equipment anchors, mechanical and electrical systems, and other ancillary components. Under this program, trained personnel visit the treatment plant sites and make recommendations for seismic and life-safety improvements based on interior and exterior inspections.

This CIP includes anticipated costs associated with this audit, as well as a 'placeholder' for the costs associated with implementation of the recommendations from this audit.

5.4.4.3 Treatment Capacity and Operations Optimization Study

Carollo recommends conducting a treatment capacity and operations optimization study. This study consists of the following:

- Primary Coagulation and Filter Operations Optimization – Surface Water.
- Greensand Filtration Optimization – Groundwater.

This study evaluates the overall pre-treatment and filter performance to identify and provide opportunities to improve filtered water quality, backwashing, and overall plant efficiency to maximize the value of the City's existing treatment infrastructure while identifying and mitigating potential treatment challenges.

This CIP includes anticipated costs associated with this study, as well as a 'placeholder' for the costs associated with implementation of the recommendations from this study.

5.4.4.4 CIP Summary

Table 5.16 presents the CIP summary table for the recommended additional studies. The table also includes an approximate service year to reflect the timing of planned projects. These studies are necessary to repair and maintain existing system facilities and to meet the needs of projected growth.

The cost estimate presented is an AACE Class 5 estimate. This is a concept screening level estimate with approximately 2 percent of the design defined, with an expected accuracy range of +100 percent to -50 percent. It is subject to change in the future.

Table 5.16 CIP Summary Table

Project	Estimated Cost	Approximate Service Year
Supply and Treatment Plant LOS Goals	$20,000.00	2019
Seismic and Life-Safety Audit Study	$60,000.00	2019
Seismic and Life-Safety Improvements	$500,000.00	
Treatment Capacity and Operations Optimization Study	$40,000.00	2019
Treatment Capacity and Operations Optimization Improvements	$250,000.00	
Total	**$870,000.00**	

Chapter 6

WATER SYSTEM ANALYSIS

6.1 Introduction

Carollo Engineers, Inc. (Carollo) evaluated the City of Scappoose's (City's) water distribution system for its ability to meet the City's reliability criteria under short-term and long-term future conditions using the medium demand projection scenario. The distribution system was evaluated for its supply and pumping capacity and reliability, the capacity of its storage facilities, and for adequate pressures and fire flow (FF) capacity using the City's updated hydraulic model.

This section discusses recommendations to eliminate each of the deficiencies identified as part of the system analysis for this Water System Master Plan Update (Plan). These recommendations form the basis of the City's Capital Improvement Plan (CIP) outlined in Chapter 8. New pipeline upsize and new pipe installation projects are recommended to ensure required FFs are available to all water mains in the service area.

6.2 Distribution System Level of Service Requirements

The City has established a level of service for its customers to provide reliable drinking water and support fire suppression activities. Comprehensive distribution system level of service requirements have been established and discussed in this Chapter:

- Pump Station Capacity – Section 6.3.
- Water Storage – Section 6.4.
- Anticipated future demands – Section 6.5.
- Available Fire Flows – Section 6.5.
- System Pressure – Section 6.7.

The City has also established level of service goals for its supplies (Chapter 4) and water quality (Chapter 5), which are important parts of providing the defined level of service to its customers.

6.3 Pumping Analysis

The City has two booster pump stations (BPS), the High Zone BPS, which provides water from the Low Zone (PZ1) to the High Zone (PZ2), and the Glenn View BPS (or Dutch Canyon Pump Station (PS]), which provides water from PZ1 to Dutch Canyon Zone (PZ4), respectively. The City's booster pumping capacities were evaluated against two criteria:

- High Zone BPS Analysis – Maximum Day Demand (MDD) + Fire Storage Replenishment. Sources shall be able to replenish depleted reservoir fire suppression storage within 72 hours while concurrently supplying MDD.
- Glenn View BPS Analysis – Peak Hour Demand (PHD). BPS to closed zones (without reservoirs) shall be able to provide sufficient flow during PHD.

Pump Stations were evaluated based on their firm capacity, defined as the largest pump out-of-service.

Carollo evaluated the water system against this criterion for each of the operating areas. The system was evaluated for all planning years. The results of this analysis are displayed in Table 6.1.

6.3.1 High Zone BPS Analysis

The analysis indicates the High Zone PS has sufficient firm pumping capacity for the short-term; however, an expansion will be required to add up to 100 gallons per minute (gpm) additional capacity to serve long-term growth in the High Zone. The City has two general alternatives to upsize the BPS:

- Install three new 25 horsepower (HP) ~220 gpm pump and motors in the existing station. The existing station has three pump pedestals (two currently used). Evaluate station hydraulics to size new (larger) pumps.
- Pump rehabilitation including mechanical, electrical, instrumentation, structural, and seismic condition.

Note, the High Zone BPS is part of the Keys Road backup power system. If Keys Road is decommissioned, a new backup generator will be required for this facility.

6.3.2 Glenn View BPS Analysis

The Glenn View BPS provides domestic supply; no fire protection is provided. The service area is a closed zone, characterized as being supplied by only booster pumps and not having storage facilities within the zone. Therefore, the Glenn View BPS must meet the PHD. The analysis shows Glenn View BPS does not have sufficient firm capacity to meet PHD starting in the short-term; however, it can meet short-term PHD with its total capacity (both pumps operating). With continued growth, by the end of the long-term planning period Glenn View BPS will be deficient for both total and firm capacity.

Note, the above analysis was based on a diurnal water use curve calculated from the entire system. A Dutch Canyon specific diurnal water use curve should be used when sizing any improvements to the BPS.

The following actions are recommended for the BPS:

- Short-term: Obtain spare parts to increase BPS reliability.
- Short-term: Update analysis using peak pumping rates from the last 5 years to determine a Dutch Canyon Zone specific PHD. Coordinate with Columbia County to update water availability for new development in the pressure zone.
- Long-term: Replace Glenn View BPS with a firm capacity of ~100 gpm from one 5 HP and two 10 HP variable frequency drive (VFD) pumps.

6.4 Storage Analysis

The City's storage requirements are a function of the City's booster pump operation, water demands, supply capacity, and FF requirements. The following sections summarize the available storage of the water system, describe the required storage components, and present recommendations to address identified storage deficits.

6.4.1 Service Areas

For storage analyses, the City's distribution system was divided into two "operating areas" based on its reservoirs. The two operating areas are as follows:

1. Low: Includes the PZ1 as well as PZ4, which is served by PZ1 via the Glenn View PS.
2. High: Includes PZ2 and Intermediate Zone (PZ3). PZ3 is fed by two pressure reducing valves (PRVs) from PZ2.

Table 6.1 Pumping Analysis Water System Plan

Service Area	High Zone (PZ2)				Dutch Canyon (PZ4)			
Planning Year	2023	2028	2033	2038	2023	2028	2033	2038
Projected Pumping Requirement								
MDD (gpm)	227	249	267	292	27	30	33	36
FF replenishment (gpm)	14	14	14	14	0	0	0	0
Total MDD + FF Needed (gpm)	241	263	281	306	27	30	33	36
PHD Needed (gpm)	NA	NA	NA	NA	93	102	110	120
Flow Required (gpm)	241	263	281	306	93	102	110	120
Available Capacity								
Pump #1 Capacity (gpm)	100	100	100	100	56	56	56	56
Pump #2 Capacity (gpm)	111	111	111	111	56	56	56	56
Fire Pump Capacity (gpm)	500	500	500	500	-	-	-	-
Firm Capacity Surplus/(Deficit) (gpm)	(30)	(52)	(70)	(95)	(37)	(46)	(54)	(54)
All Pumps Surplus/(Deficit) (gpm)	470	448	430	405	19	10	2	(8)

6.4.2 Available Storage

The City has five storage tanks with a total capacity of 3.87 million gallons (MG), as shown in Table 6.2. Note, the fifth reservoir, an 0.22 MG reservoir at Keys Road is no longer in use due to its condition. The available storage in each operating area is controlled by the elevation of the highest customer in the system and the hydraulic grade level (HGL) required to serve that customer with a pressure of at least 20 pounds per square inch (psi) in the case of a fire or other emergency, or 35 psi under normal conditions. Table 6.2 shows the highest service elevation and the amount of available storage meeting the 35 psi and 20 psi requirements in each operating area. The maximum headloss to the customer was not included in the calculation as the system experiences minimal headloss as observed during the FF tests. All storage is available; the City has no dead storage currently, which is discussed in the next section.

6.4.3 Storage Components

The five components of storage are listed below and are illustrated in Figure 6.1. These components should be considered for any water system:

1. Operational storage.
2. Equalizing storage.
3. Fire Suppression storage.
4. Emergency storage.
5. Dead storage.

Operational and equalizing storage must be available to all customers at a residual pressure of at least 35 psi under PHD flow conditions. Standby and fire suppression storage must be available to all customers at a residual pressure of at least 20 psi under MDD. Dead storage is the volume in the tank that cannot be used to serve the highest customer in the water system with a pressure of at least 20 psi. Thus, there are two blocks of available storage: the volume of storage available to all customers with a pressure of at least 20 psi, and the volume of storage available to all customers at a pressure of at least 35 psi.

6.4.3.1 Operational Storage

Operational storage is the band of storage within each reservoir that is utilized during periods of average demand. It is typically estimated based on the volume of water each reservoir drops prior to calling on the supply sources, and is measured as the volume of water stored between the pump call-off and pump call-on levels. Operational storage was set at two feet (ft) based on input from City operations staff.

Table 6.2 Available Storage

Service Area	Low	Low	High	High
HGL	200	200	430	430
Facility	KEYS 1.0 MG	KEYS 2.0 MG	BELLA VISTA 0.3 MG	BELLA VISTA 0.37 MG
Reservoir Information				
Date Installed	1967	2005	1967	2003
Geometry	Circular	Circular	Elevated	Elevated
Storage Capacity (gal)(1)	1,372,351	1,846,274	282,027	341,159
Elevation of Overflow (ft)	200	200	430	430
Base of Tank (ft)	176	179	400	394
Nominal Diameter (ft)	98	123	40	40
Pump out of Reservoir?	No	No	No	No
Available Storage				
High Service Elevation (ft)	107	107	323	323
HGL Required by Highest Customer (35 psi) (ft)	188	188	404	404
HGL Required by Highest Customer (20 psi) (ft)	153	153	369	369
Existing Storage Above 35 psi HGL (gal)	676,000	1,080,000	246,000	249,000
Percent of Storage Above 35 psi HGL	49%	58%	87%	73%
Existing Storage Above 20 psi HGL (gal)	1,372,000	1,846,000	282,000	341,000
Percent of Storage Above 20 psi HGL	100%	100%	100%	100%

Note:
(1) gal: gallons.

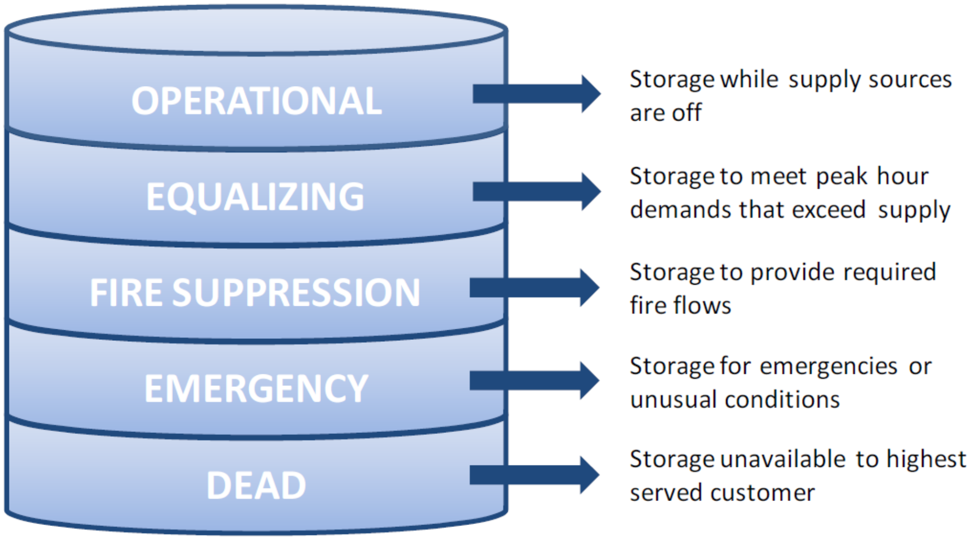


Figure 6.1 Storage Components

6.4.3.2 Equalizing Storage

Equalizing storage is the volume needed to satisfy PHD. It must be available at 35 psi to all service connections. The required equalization storage was developed using 2016 supervisory control and data acquisition (SCADA) data as shown in Figure 6.2. Based on this analysis, the City defined equalization storage as 25 percent of MDD.

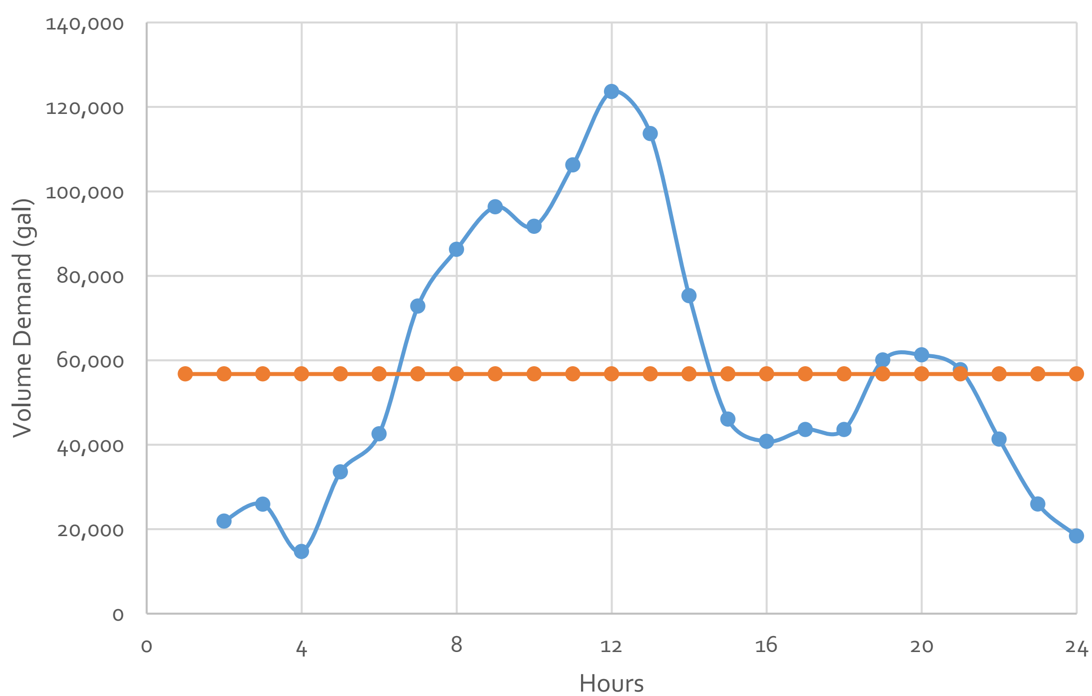


Figure 6.2 Equalizing Storage SCADA Analysis

6.4.3.3 Fire Suppression Storage

The City maintains separate storage for fire suppression storage and emergency storage. Fire suppression storage is the volume of storage required to deliver FFs as prescribed by local fire protection authorities, while maintaining a minimum pressure of 20 psi throughout the entire water system. Since a fire can occur at any time, the fire suppression storage must be in addition to the emergency, equalizing, and operational storage. The maximum fire suppression volume required by Operating Area are:

- Low Operating Area: 0.63 MG based on 3,500 gpm for 180 minutes (3 hours) FF requirement.
- High Operating Area: 0.06 MG based on 1,000 gpm for 60 minutes (1 hour) FF requirement.

6.4.3.4 Emergency Storage

Emergency storage is the volume of storage required to supply reasonable system demands during a system emergency, such as disruption of the water supply. Disruptions could be caused by transmission pipeline or equipment failure, power outage, valve failure, or other system interruptions. The computation of emergency/standby storage requirements includes consideration of reasonable system disruptions that can be expected to occur within normal planning contingencies, and does not consider major system emergencies, such as earthquakes that result in shutdown of water supplies and multiple distribution system breaks. These types of emergencies should be covered under emergency system operation planning. The City requires emergency storage volume equivalent to two times the average day demand (ADD).

6.4.4 Storage Analysis

The storage analysis compares available to required storage. The storage analysis for each operating area and planning year are presented in Table 6.6. Under total required storage, the volume required above the 35 psi HGL is the sum of operational and equalizing storage. The volume required above the 20 psi HGL is the sum of operational storage, equalizing storage, fire suppression, and emergency storage.

Approximately 1.5 MG of total new storage capacity will be needed by 2038. Increased equalization and emergency storage with development are major drivers for the need for additional storage capacity. It is anticipated that the Low Operating Area will have a shortage of 1.3 million gallons per day (mgd) by 2038. The High Operating Area will experience a shortage of 0.05 mgd.

6.4.5 Storage Improvements

To meet future storage needs, the City will develop a new 2 MG reservoir on the Keys Road Water Treatment Plant (WTP) site, creating a third Low Zone reservoir at the site. The High Zone BPS, with recommended improvements, will be used to provide emergency supplies to the High Zone. High Zone storage needs will be met from Low Zone via improved BPS pumping capacity, therefor no new storage is recommended in this zone. Note, sizing includes 0.5 MG for future growth beyond the planning period.

A second Low Zone reservoir site and elevated storage were considered, though not recommended. Constructing a ground reservoir to the south of the service area is limited by hydraulics and there are seismic concerns with the steep hill slope at the appropriate site elevation (170 ft to 180 ft). An elevated storage tank is feasible; however, it would likely be four or five times more expensive per gallon than ground storage, require additional land acquisition, and may not have acceptable aesthetics to the community. Therefore, it is not recommended.

6.5 Hydraulic Model Development

The City's hydraulic model is the primary tool for evaluating the City's distribution system. The model evaluates how the City's water infrastructure handles future demands and verifies that recommended improvements will eliminate system deficiencies.

As part of this project, a hydraulic model was developed in InfoWater by Innovyze and calibrated. The hydraulic model was developed using the data provided by the City for the various elements of the hydraulic model described below. A summary of the model is presented in this section.

6.5.1 Elements of the Hydraulic Model

The following provides a brief overview of the various elements of the hydraulic model and the required input parameters associated with each:

- Junctions: Junctions are often located where pipe sizes change, where pipelines intersect, or where water demands are applied and are represented by junctions in the hydraulic model. Required inputs for junctions include service elevation and water demands.
- Pipes: Water mains are represented as pipes in the hydraulic model. Input parameters include length, diameter, roughness coefficient, and whether or not the pipe includes a check valve (i.e., does not allow reverse flow). The recent development at Vinterra was added to the model.
- Tanks: Water tanks are included in the hydraulic model as cylindrical. All of the City's storage reservoirs are above ground cylindrical, and made of steel and concrete. Required input parameters for cylindrical tanks include bottom elevation, maximum level, initial level, and diameter.
- Water Sources (Fixed Head Reservoirs): For water distribution system modeling, fixed head reservoirs are used to represent a water source with a constant HGL. Typically, fixed head reservoirs are used to represent water sources, such as groundwater supplies or a regional transmission line.
- Pumps: Multiple pump types are included in the hydraulic model. Input parameters for pumps include pump curves and operational controls.
- Valves: A number of different valves, such as PRVs and level indicators, are represented in the hydraulic model. Required input parameters for valves include diameter, operational controls, and other settings or headloss curves depending on the type of valve.
- Demands: Water demands are applied at specific junctions in the hydraulic model. Up to ten different demands can be assigned at a particular junction.

- Fire Flows: FFs are simulated by assigning a fire demand to certain junctions in the model based on land use. The modeling software will then run a system-wide FF analysis, in which each junction with an assigned FF will be analyzed and a residual pressure will be computed. This eliminates the need to manually run FFs throughout the system and increases the number of junctions that can be analyzed.

6.5.2 Model Development

The City's hydraulic model was developed using the City's Geographic Information System (GIS) data. The GIS was based on May 2017. The all-pipes model contains 2,303 nodes and 2,432 pipes. In addition, there are 4 tanks, 7 well and supply sources, 2 PRVs, and 6 pumps. The model is shown in Figure 6.3. This section summarizes all updates performed in the hydraulic model for the purpose of this system analysis.

6.5.2.1 Pressure Zones

The pressure zone boundaries are verified and updated when necessary in the hydraulic model. Pipes are permanently closed or open to match the updated pressure zone configuration of the water system. The "zone field" was used to assign pressure zones for both pipes and junctions.

6.5.2.2 Junctions and Elevation Data

Junctions were added in the model at the location of all hydrants used during the field hydrant tests for the purpose of model calibration and other significant areas. Junction elevations were interpolated from two-foot interval contour data provided by the City. Using contour data, ground elevations are extracted and assigned to all junctions in the model. Pipe elevations were associated with their connecting junctions. Ground elevations were used for all elevation data, as actual pipe and meter elevations were unknown.

Elevation for storage tanks, pumps, and PRVs were assigned based on data provided by the City during the development of this Plan.

6.5.2.3 Pressure Reducing Valves

PRVs are used to reduce the pressure as water flows between different pressure zones. PRVs were assigned a valve elevation, valve diameter, valve pressure zone, and valve setting based on information provided by the City.

Table 6.3 Operational Storage

Service Area	Low			High	
HGL	200	200	200	430	430
Facility	KEYS 1.0 MG	KEYS 0.2 MG	KEYS 2.0 MG	BELLA VISTA 0.3 MG	BELLA VISTA 0.37 MG
Geometry	Circular	Circular	Circular	Elevated	Elevated
Nominal Diameter (ft)	98	15	123	40	40
Volume/Height (gal/ft)	56,429	1,322	88,891	9,401	9,401
Operating Band (ft)	2.0	2.0	2.0	2.0	2.0
Operating Volume (gal)	113,000	3,000	178,000	19,000	19,000
Percent of Total Storage	8%	13%	10%	7%	6%

Table 6.4 Equalizing Storage Calculations

Service Area	Low		High	
Planning Year	2028	2038	2026	2036
MDD (gpd)[(1)]	2,037,000	2,548,000	359,000	420,000
Equalizing Storage (%)	25%	25%	25%	25%
Required Equalizing Storage (gal)	509,000	637,000	90,000	105,000
Percent of Total Storage	16%	20%	14%	17%

Note:
(1) gpd: gallons per day.

Table 6.5 Standby Storage Calculations

Service Area	Low		High	
Planning Year	2028	2038	2028	2038
ADD (gpd)	1,167,000	1,494,000	196,000	233,000
Emergency Storage Factor	2	2	2	2
Standby Storage (gal)	2,334,000	2,988,000	392,000	466,000
Percent of Total Storage	72%	92%	63%	75%

Table 6.6 Required Storage Components

Service Area	Low	Low	High	High
Service Area	Low	Low	High	High
Planning Year	2028	2038	2028	2038
Available Storage (gal)				
Total Storage	3,218,626	3,218,626	623,186	623,186
Meeting 35 psi Requirement	1,756,000	1,756,000	495,000	495,000
Meeting 20 psi Requirement	3,218,000	3,218,000	623,000	623,000
Required Storage Components (gal)				
Operational Storage	294,000	294,000	38,000	38,000
Equalizing Storage	509,000	637,000	90,000	105,000
Fire Suppression Storage	630,000	630,000	60,000	60,000
Emergency Storage	2,334,000	2,988,000	392,000	466,000
Required Storage (MG)				
To meet 35 psi Requirement	803,000	931,000	128,000	143,000
To meet 20 psi Requirement	3,767,000	4,549,000	580,000	669,000
Storage Surplus/(Deficit) (MG)				
Meeting 35 psi Requirement	953,000	825,000	367,000	352,000
Meeting 20 psi Requirement	-549,000	-1,331,000	43,000	-46,000
Surplus/(Deficit) (MG)	(549,000)	(1,331,000)	43,000	(46,000)
Final Surplus/ (Deficit) (MG)	(549,000)	(1,331,000)	43,000	(46,000)

6.5.2.4 Pipes

Pipes included in the model were checked against the most recent City GIS data and diameters were updated accordingly.

6.5.2.5 Storage Tanks

All of the City's tanks are modeled as cylindrical tanks. Storage tank dimensions, such as diameter, height, bottom elevation, were assigned based on the latest data provided by the City.

6.5.2.6 Supplies

The City has three existing water supply sources, one surface source (Keys Road) and four wells (Miller Road and Dutch Canyon). Supplies were modeled as fixed head reservoirs in the hydraulic model. Surface water supply HGLs were estimated based on drawings provided by the City. Flow control valves were added downstream of a source in the hydraulic model to limit the supply to its maximum flow, as indicated by the City.

The City operates two water treatment facilities, Miller Road and Keys Road. Water flows via gravity from the surface water intakes and via a well pump from Dutch Canyon to the Keys Road Treatment plant and by gravity fills the Keys Road tanks. Miller Road is run using a single treatment train at a time (~430 gpm) due to limited well capacity.

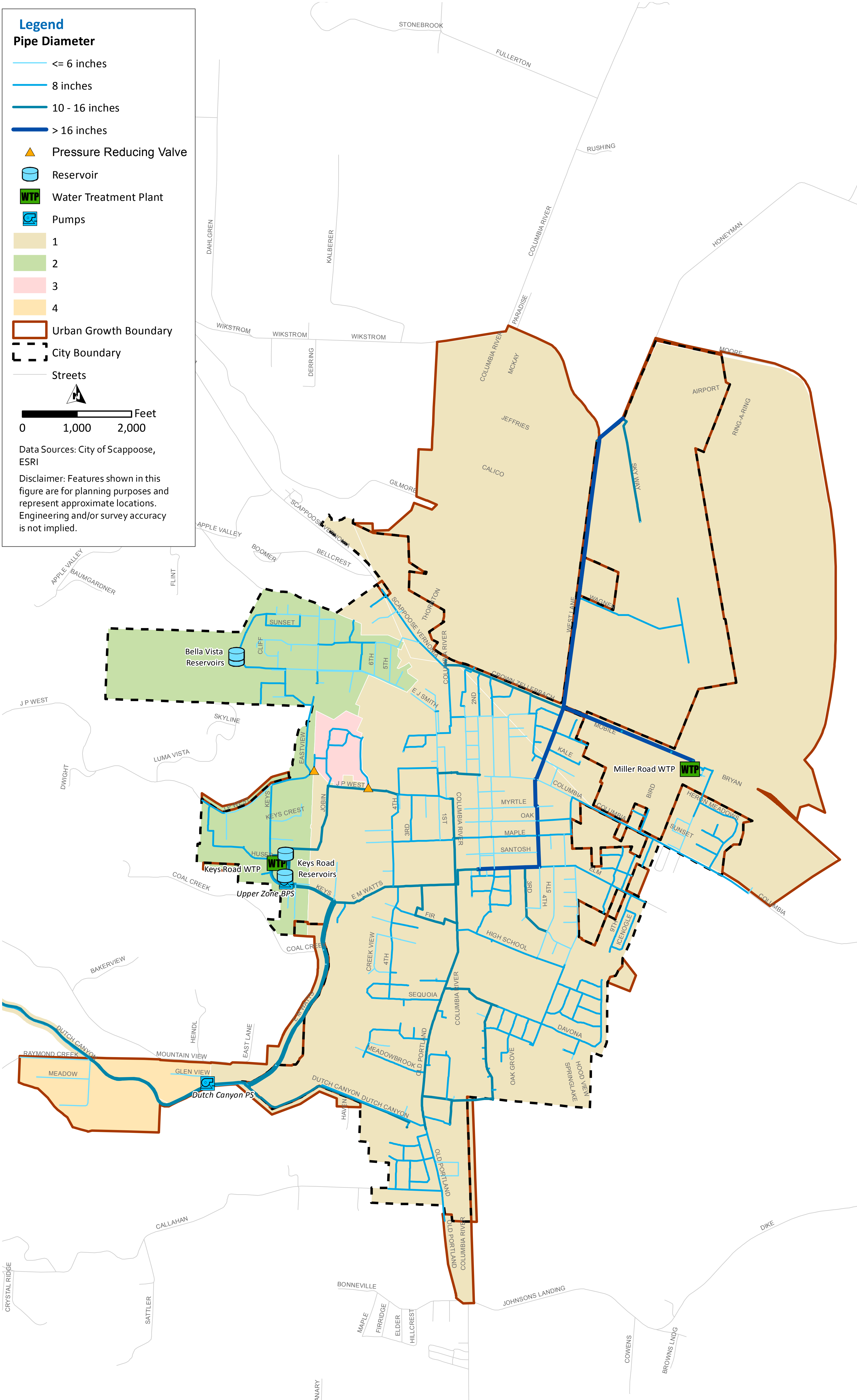


Figure 6.3 System Overview

6.5.2.7 Operational Settings

Operational settings for tanks, reservoirs, pump stations, and valves were developed based on operational data developed by the City. The supplies open and close based on the Keys Road tank elevations. The City indicated the preferred supply order of: surface water, then Dutch Canyon, then Miller Road wells. The High Zone booster pumps turn on and off based on the level at the Bella Vista tanks. Logical based controls were added to the hydraulic model.

6.5.3 Demand Allocation Process

6.5.3.1 Process Overview

The model was developed with the medium demand projections presented in Chapter 3 – Water Requirements, which provides demands by customer type for each pressure zone. The demand allocation process spatially distributed these future demands to the model's many nodes. Each node represents the demands from nearby customers that may include multiple customer types (e.g., commercial and residential). Demand is allocated based on the number and class of customers contributing to each model node.

The land use of the contributing area, in acres, to each node was calculated using GIS. The demand projections were developed based on accounts, not area, therefore the demands were converted to a demand per acre factor. Using the demand per acre factors, the projected demands were calculated for the contributing area to each node.

6.5.3.2 Demand Allocation

The demand allocation was based on the land use within these contributing areas. Future demands were also allocated based on land use. For planning purposes, the future service area considers supplying additional annexation surrounding the airport. Future scenarios consist of 2026 (short-term) and 2036 (long-term) planning years. Note, there is additional 0.1 mgd of demand when considering planning years 2028 and 2038. Modeled results were considered characteristic of short-term and long-term system operation, and the model was not re-run using 2028 and 2038 demands.

The demands were projected by customer type. Customer types included: general residential, commercial, industrial, manufactured home, and public lands. Each customer type corresponds to a particular land use type. General residential demands correspond to general residential and suburban residential land use. The commercial customer type corresponds to commercial land use and the manufactured home customer demands were assigned to the manufactured home land use type. The public lands customer type corresponds to the public lands land use and the industrial customer type corresponds to the industrial land use type. Open spaces were not allocated demands. Water loss and unbilled unmetered demands were allocated uniformly across the system. No large consumer demands were assigned in the future.

Future demand per acre factors were calculated for the demand anticipated in addition to existing demand by 2026 and 2036. Future demands were allocated on top of existing demands to nodes.

Each node represents the demands from nearby customers, which may include customers from multiple classes. The contributing area of each land use type to each model node on the parcel scale was calculated using GIS. Automated GIS tools initially assigned parcels to each model node. The results of the automated analyses were reviewed and some parcels were reassigned to better represent the source of water for the customers. Commonly, undeveloped or vacant areas were reassigned to the nearest potential system connection to approximate the impact of expansion on the existing system. Additionally, parcels bordering multiple mains were reviewed and reassigned when necessary.

The node demand was calculated by multiplying the demand per acre factors by number of acres for each land use type contributing to the node.

The resulting demand allocation does not establish the actual water use for individual customers; rather it represents a typical water use based on large groups of customers. Similarly, the actual site of development or redevelopment is not considered, rather future demands are spread across a large area that the City has established as vacant or having the potential for redevelopment.

Note, the demands presented in this section were developed for planning purposes and should not be used for permitting or design of development-scale projects.

Note:

- Demand Collection 1 – Existing Demands.
- Demand Collection 2 – Annex Demands.
- Demand Collection 3 – 2026 Demands.
- Demand Collection 4 – 2036 Demands.
- Demand Collection 5 – Water Loss.

6.5.4 Fire Flows

FF demand requirements were assigned in the model. The quantity of water available for firefighting establishes an important level of service for a water system. The City's established criteria for FF were used in the hydraulic model. The following criteria are established minimum requirements:

- 1,000 gpm for all residential areas of the City.
- 2,000 gpm for non-residential areas of the City.
- 3,500 gpm for the annexation / airport employment.

Figure 6.4 shows the general FF requirements throughout the distribution system. Note, specific developments may exceed the general requirements as established by the Fire Marshal on a site-specific basis.

6.6 Hydraulic Model Calibration Overview and Methodology

Calibration is the process of comparing model simulation results to actual field data, and making corrections and adjustments to the model to achieve a loose agreement between model predictions and field measured data. This section describes the different steps of the hydraulic model calibration.

6.6.1 Model Calibration Overview

The purpose of the water system hydraulic model is to estimate, or predict, how the water system will respond under a given set of conditions. One way to test the accuracy of the hydraulic model is to create a set of known conditions in the water system and then compare the results observed in the field against the results of the hydraulic model simulation using the same conditions. Field flow tests can verify data used in the hydraulic model and yield a greater understanding of how the water system operates.

Field testing can help identify errors in the data used to develop the hydraulic model, or show that a condition might exist in the field that is not otherwise known. Valves reported as being open might actually be partially closed or closed (or vice versa). An obstruction could exist in a pipeline, or pressure settings for a PRV may be different from noted. Field-testing can also correct erroneous model data such as incorrect pipe diameters or connections. Data obtained from the field tests can be used to determine appropriate roughness coefficients for each pipeline, as roughness coefficient can vary with age, pipe material, and construction quality. Other parameters can also be adjusted to generate a calibrated model.

The calibration process for the City's hydraulic model consisted of two parts: a hydrant flow test calibration, or micro calibration, and an Extended Period Simulation (EPS) calibration, or macro calibration. The following sections describe both calibration steps.

6.6.2 Macro-Calibration: Pressure

The macro calibration process involved several steps to ensure that the model produces reasonable results:

- Facility Characteristics. Hydraulic model results from each booster pump station, supply source, and valve are compared to the known conditions to verify that the facilities produced results comparable to expectations. This identified problems with elevation, connectivity, as well as operational controls.
- Transmission Main Connectivity. The connectivity tool in InfoWater was used to verify the transmission mains were connected. Problems found using the connectivity locator were reviewed to determine whether adjustments were needed to the pipe network. Output reports of pipe flow characteristics such as head loss per thousand feet and velocity were also used to locate potential network problems.

Minor issues were identified in the macro calibration process and corrected. The resulting model was then calibrated using the field hydrant tests.

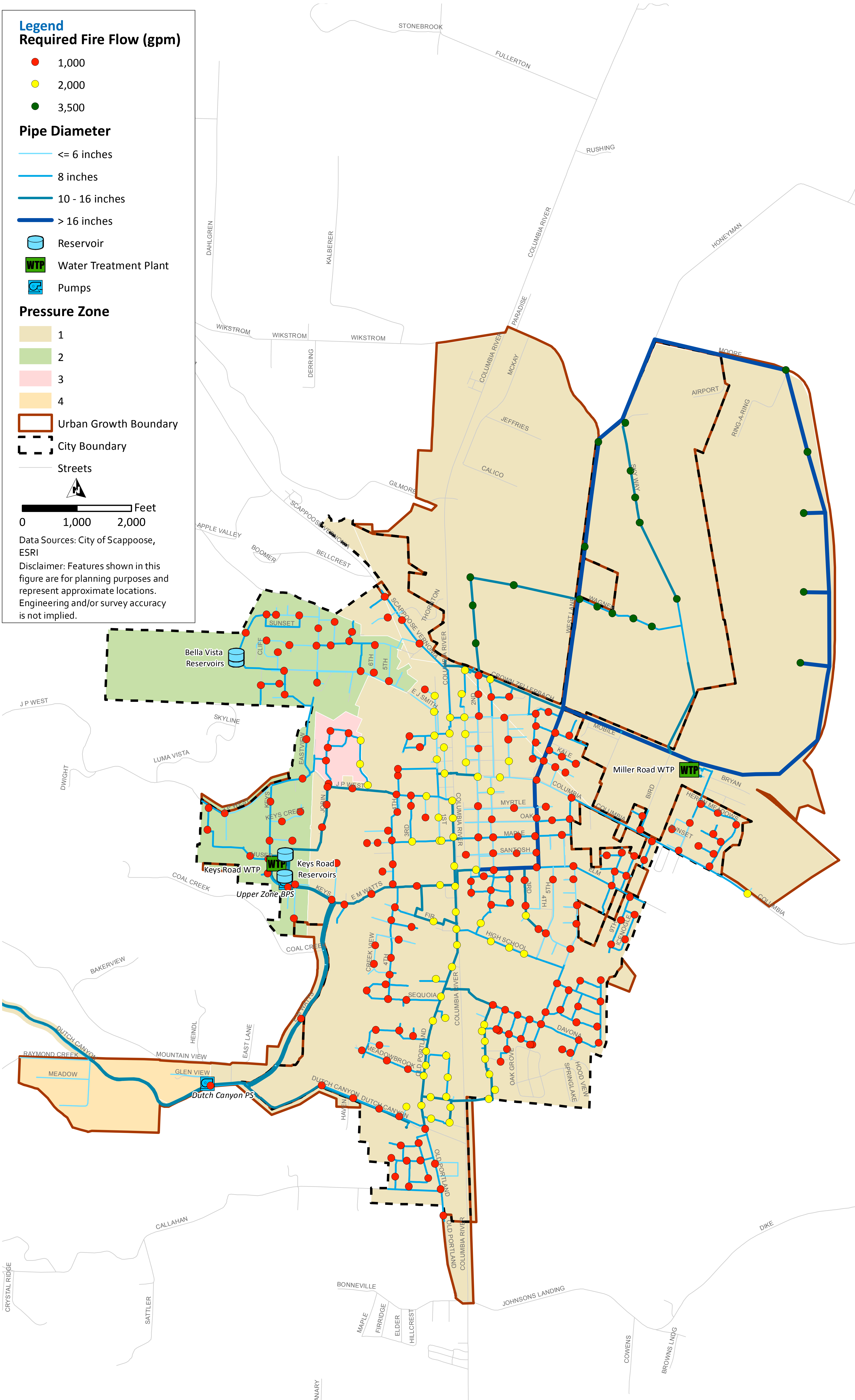


Figure 6.4 Fire Flow Requirements

6.6.3 Micro-Calibration: Hydrant Flow Test Calibration

6.6.3.1 Hydrant Flow Test Calibration Overview

During ADD conditions, roughness coefficients have a relatively small effect on operation of the distribution system due to low velocities. As flows increase in the system on higher demand days or during FFs, velocities within pipelines increase leading to higher system head losses.

The hydrant flow tests stressed the distribution system by creating a differential between the HGL at the point of hydrant flow and the system HGL at neighboring hydrants. This HGL differential increases the effect of the roughness coefficients on system losses. The calibration to hydrant flow tests are intended to develop a calibrated hydraulic model by closely matching model-simulated pressures to field pressures under similar demand and system boundary conditions. The primary varied parameter for this calibration is the pipeline roughness coefficient; although other parameters can also be adjusted as calibration results are generated such as pump controls and curve of operation of other automatic control valves.

The model is calibrated by simulating the hydrant flow test and adjusting settings and parameters to match the field measured pressures under similar demand and system boundary conditions. For the monitoring hydrants, the results are considered acceptable if model pressures are within 10 psi or have a 10 percent difference to both the static and residual field data. Model pressures within 5 psi or 5 percent of the field measurements are considered very good.

The Hazen-Williams roughness coefficient, or C-factor, is a function of pipeline material, diameter, and age. In addition, for simplicity in the model, minor losses were not applied at fittings, and instead losses at fittings were incorporated into (slightly higher) C-factors. Hydrant test calibration refines the initial estimation of the value of roughness coefficients that best represent current conditions within the City's distribution system. The roughness coefficients should be adjusted only within the accepted roughness coefficient range of $80 < C < 130$.

If the model is unable to match the calibration results within the acceptable range of roughness coefficient values for a given pipeline material and age, there may be cause for further investigation of a previously unknown field condition. Examples of conditions that can arise during hydraulic model calibration include closed valves, partially closed or malfunctioning valves, extreme corrosion within pipelines or connectivity, and diameter errors in GIS layers, record drawings, or diurnal patterns of large water users.

A single static pressure scenario was created in the model. For each hydrant flow test, a residual pressure scenario was also created in the hydraulic model. Each residual pressure scenario comprises of a different demands dataset in order to produce the model conditions similar to the field conditions during each hydrant flow test. All other datasets are the same for all the scenarios. This set-up makes it easy to check all the calibration points at the end of the calibration process in order to make sure that any adjustments made to one zone did not affect the calibration points in the adjacent pressure zones.

6.6.3.2 Hydrant Flow Test Data

The City conducted a hydrant flow test program throughout the entire system on February 5th, 2018 in order to gather recent and detailed information for calibration of the updated hydraulic model. The City conducted seven flow tests in two of the system's four pressure zones. Typically, when conducting hydrant flow tests, there should be a minimum 10 psi pressure drop (static pressure minus residual pressure) to assist in calibration, where values less than this are within the range of error seen in a planning-level model. However, a 10 psi pressure drop was difficult to achieve for the City due to the redundant nature of the City's distribution system. Therefore, calibration at these sites was considered approximate.

Figure 6.5 shows the locations of all hydrant flow tests performed in the distribution system. Appendix K documents the complete hydrant flow test program and test report and system operating data.

The key data collected during the field tests includes:

- Test Location (Fire Hydrant Identification (ID), Static Pressure Reading Address/ID): It is very important to locate the exact nodes in the model where the fire hydrant test is performed and results read.
- Test Time: Tests were performed between 9:00 in the morning and 11:30 in the morning. As noted previously, demands fluctuate throughout the day and therefore need to be adjusted in the model to reflect the test time.
- Hydrant Flow: The hydrant flow directly affects head losses through the system, and therefore the residual pressure.
- Static and Residual Pressures: These are the values that the model needs to match within the criteria.
- SCADA Data for pumps and tanks: Tank levels and pump operations at the time of the hydrant test are set in the model for each test case.
- Comments: Certain comments from the operators during the test are helpful during calibration.

6.6.3.3 Hydrant Flow Test Calibration Results

Calibration to hydrant flow tests is conducted individually in order to specifically represent the conditions of the system at the time of the test. Therefore, numerous simulations are performed during the calibration phase. Adjustments are made to the model between runs to minimize the differences between the model and the field measured results. SCADA data on tank levels, PS, and inlet flows are available in Appendix K.

The results of the calibration are summarized in Table 6.7 and Figure 6.6. The model was calibrated within 10 psi or 10 percent of the field-measured pressures for each hydrant test site. Appendix K presents a more detailed summary of the calibration results, including the location, time, and results of each field test conducted and corresponding hydraulic model results. All sites are within the calibration standards and most of the sites are calibrated within 5 psi, which is considered a very good calibration.

Figure 6.6 summarizes all calibration point results on a 1 to 1 plot. A linear regression analysis was performed on the data comparison. The linear regression curve obtained from this comparison intercepts at zero with a slope near 1 and a percentage of determination (R2) of 98.1 percent, the calibrated model matches closely the field data for the range of test conditions.

6.6.4 Model Recommendations

The calibrated hydraulic model provides an excellent tool for evaluating the distribution system. The model should be updated periodically to maintain reasonable prediction of water system conditions. An update would include incorporating main replacements and improvements, adding new service areas, incorporating operational changes to the tanks and pumps, adjusting PRV settings, and adjusting demands to match demand projections and land use. As part of this periodic update, hydrant flow tests should be conducted to verify the accuracy of the model and aid in monitoring system changes. Additionally, FF tests should be conducted to validate model results for new developments in areas with low pressures or high head loss.

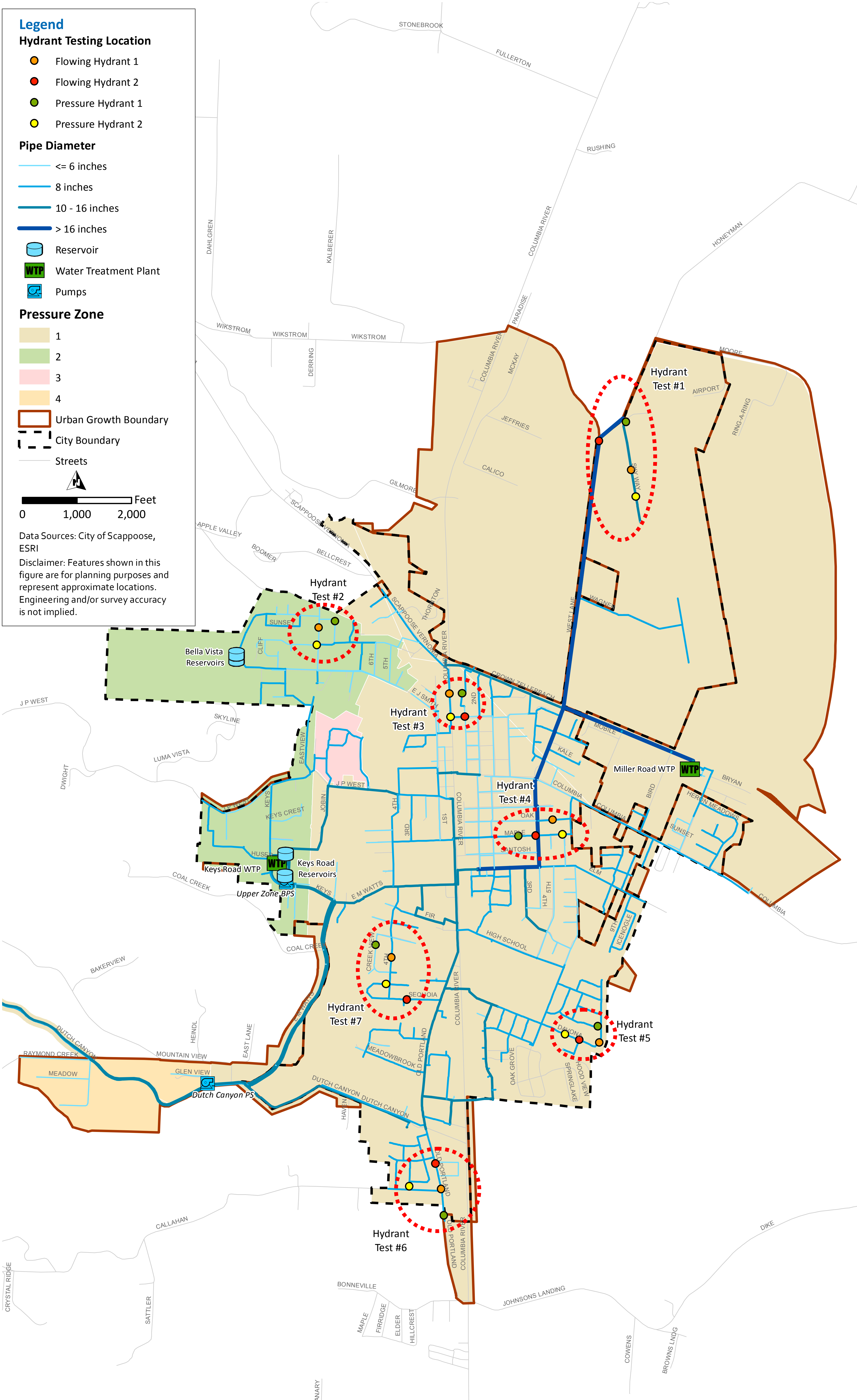


Figure 6.5 Hydrant Flow Calibration Test Sites

6.7 Distribution System Analysis

The hydraulic model was used to evaluate the distribution system under future demand conditions. The distribution system was evaluated against four performance criteria. Areas not meeting the criteria are considered deficient and system improvements are identified to achieve the required level of service.

6.7.1 Evaluation Criteria

The evaluation criteria are from the City's policies and criteria presented in Section 2. The distribution system was evaluated for the following criteria:

1. High ADD Pressure. Maximum recommended pressure is 120 psi during ADD.
2. Low PHD Pressure. Minimum allowed pressure is 35 psi during PHD.
3. High Velocity. Maximum allowed velocity is 8 ft per second (ft/s) during PHD. Maximum allowed velocity if 10 ft/s during MDD + FF.
4. Available FF. During FF during MDD conditions, system pressures must remain above 20 psi.

6.7.2 Identified Deficiencies

6.7.2.1 High ADD Pressures

The City has a goal of limiting pressures to 120 psi to aid in operation and maintenance (O&M) of the distribution system. Customers with meter pressure exceeding 80 psi are required to have individual PRVs per the International Plumbing Code. The model was run during 2026 ADD to identify areas with high pressures, which is the planning year where demands will be the lowest and therefore pressures will be the highest. Areas with high pressures were largely in the High Zone as elevations decreased down the hill, as shown in Figure 6.7. These locations have elevations of approximately 50 ft while the HGL of the zone is approximately 430 ft. Note, it is anticipated that the pressures would exceed those shown on Figure 6.7 during times of lower water usage.

To reduce O&M risk, the following Alternatives are recommended to reduce high pressures:

- Alternative 1 - Split the zone north of NW Ej Smith Road from ~100 psi (400 ft HGL) to 70 psi (330 HGL). Two PRV stations are recommended for redundancy and looping: Belle Vista Drive and NW Peak Road.
- Alternative 2 - Serve NW 7th and east from the Low Zone. For redundant supply lines, a PRV from the High Zone or a Scappoose creek crossing will be required. Some replacement of the 4 inch steel will likely be required to achieve required FFs.

The new zone is anticipated to be constructed in the long-term or when required to serve new development. Alternative 1 was considered the preferred alternative and used in the CIP, as it can be implemented without new piping.

Table 6.7 Calibration Summary Results

Test No.	Hydrant Number	Model Junction ID	El. (ft)	Flow (gpm)	Field Results		Model Results		Comparison			
					Static Pressure (psi)	Residual Pressure (psi)	Static Pressure (psi)	Residual Pressure (psi)	Static Pressure Diff (psi)	Residual Pressure Diff (psi)	Static Pressure Error (%)	Residual Pressure Error (%)
1	119	J3576	57.46	698	63	57	61	57	2.2	0	3.4%	0.5%
	117	J6270	57.48	748	60	54	61	57	-0.8	-3	-1.4%	-6.3%
	118	J3566	57.4	0	64	60	61	57	3.2	3	4.9%	4.5%
	150	J7968	57.27	0	61	57	61	57	0.1	0	0.1%	0.4%
2	331	J4746	161.71	962	113	101	112	103	0.9	-2	0.8%	-2.4%
	345	J6190	135.55	0	123	116	123	117	-0.5	-1	-0.4%	-0.8%
	330	J4274	153.16	0	118	110	116	110	2.0	0	1.7%	0.4%
3	315	J10216	61	662	63	56	59	56	3.7	0	5.9%	0.2%
	101	J8220	64.82	715	55	49	58	55	-2.6	-6	-4.8%	-11.2%
	520	J6780	61.91	0	62	58	59	56	3.1	2	5.0%	2.9%
	312	J4378	63.88	0	59	56	58	55	1.0	1	1.6%	1.9%
4	224	J4976	47.24	698	65	59	65	63	-0.2	-4	-0.3%	-7.2%
	226	J3500	51	764	60	55	64	62	-3.6	-7	-6.0%	-12.7%
	216	J4060	53.38	0	65	62	63	61	2.4	1	3.8%	1.5%
	227	J3692	39.56	0	69	65	69	67	0.5	-2	0.7%	-3.1%
5	260	J1676	15.87	780	74	69	79	68	-4.7	1	-6.4%	0.8%
	261	J7344	18.57	780	73	64	78	67	-4.6	-3	-6.3%	-5.1%
	259	J2384	14.16	0	82	75	79	71	2.5	4	3.1%	4.9%
	262	J10214	21	0	78	70	77	68	1.5	2	1.9%	3.4%
6	449	J3444	69.57	624	57	47	55	46	1.5	1	2.7%	1.7%
	448	J3430	69.91	643	54	44	55	46	-1.3	-2	-2.4%	-5.3%
	450	J3458	67.6	0	58	51	56	47	1.7	4	2.9%	7.7%
	510	J5438	77.8	0	55	46	52	44	3.1	2	5.6%	5.0%
7	521	J2896	64.67	584	58	53	58	53	0.4	0	0.7%	-0.3%
	426	J2690	65.37	662	54	49	57	53	-3.3	-4	-6.1%	-7.9%
	419	J3184	63.72	0	59	56	58	54	1.0	2	1.7%	3.3%
	423	J6154	63.3	0	59	54	58	54	0.8	0	1.4%	0.4%

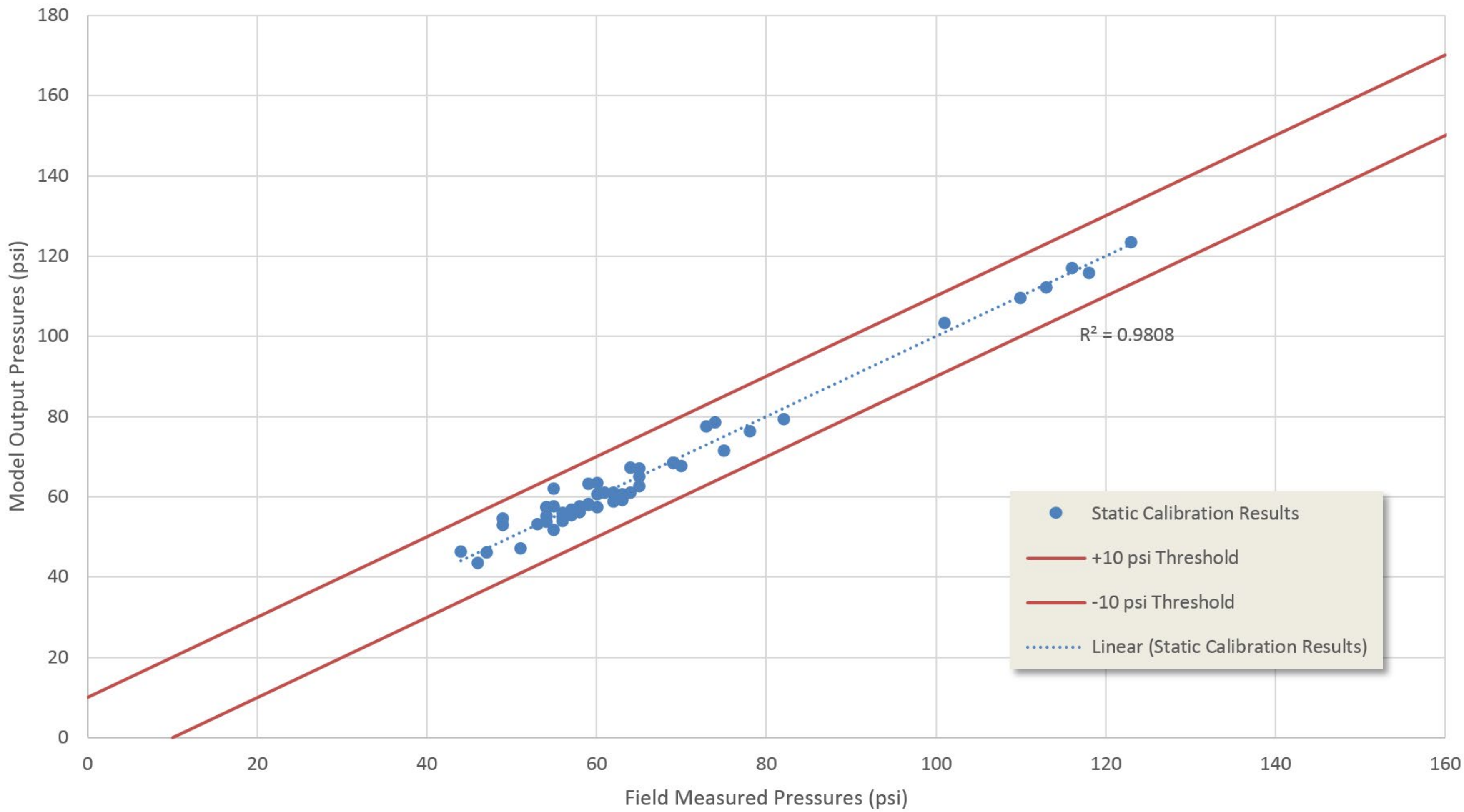


Figure 6.6 Static Calibration Results

6.7.2.2 Low PHD Pressure

PHD conditions were simulated for each planning year to identify areas with operating pressures below 35 psi. Low pressures were identified near the City's storage reservoirs and on SW Dutch Canyon Road east of SW Em Watts Road, as shown in Figure 6.8, corresponding to areas of high elevations. The figure provides results for the long-term, which was the planning year of highest demand and therefore lowest pressure. Only nodes with demands in the model are plotted.

The City has not received any customer complaints regarding low pressures near the reservoirs and no improvements were identified for these areas. Pressures at the SW Dutch Canyon Road are anticipated to improve slightly with the capacity improvements presented below.

6.7.2.3 High Velocity during PHD

The City's goal is to maintain velocities under 8 ft/s in distribution pipes during the PHD. No pipes were found to exceed the velocity criteria in any planning year. Therefore, no improvements are recommended.

6.7.2.4 Available Fire Flow

The City criterion requires FFs to be met while supplying MDD and maintaining 20 psi throughout the distribution system. FFs are typically the largest flows a system experiences and are often a major factor in pipe sizing and configurations. The hydraulic model was used to systematically simulate a fire at all model nodes representing fire hydrants for each of the planning years.

During the FF analysis, reservoirs were set to the bottom of the fire pool, at a level of one foot. This is often much lower than typical operating levels. Therefore, locations that may have sufficient pressure and flow during annual hydrant testing may be deficient in the model at these lower reservoir levels.

For each of the planning years, deficient nodes that cannot provide required FFs while maintaining system pressures everywhere else in the system above 20 psi are shown in Figure 6.9. Short-term FF deficiencies are shown in orange, those in both the short-term and long-term are shown in red. Also shown in Figure 6.9 are pipe where velocities will exceed the City's criteria of 10 ft/s. These pipes include:

- Pipes less than or equal to 4 inches supplying any hydrant.
- Dead-end 6 inch pipe supplying any hydrant.
- Dead-end 8 inch pipe supplying a non-residential hydrant.

There are FF deficiencies throughout the system, primarily on dead-end mains, areas of high elevation, and in the airport expansion area. All deficiencies occur in the short-term and worsen by the end of the long-term planning period with the exception of select hydrants in the airport expansion area which are resolved due to planned future piping. Additional looping in the long-term is also recommended to facilitate O&M.

6.7.3 Distribution System Redundancy

The City has expressed concern regarding losing supply to a portion of the High Zone due to a single supply line near an active landslide and a creek crossing. Figure 6.10 provides details regarding three possible alternatives for additional redundancy to this area:

- Alternative 1 – Pipe replacement / new pipe from the intersection of NW Eastview Drive to the intersection of NW View Terrace and NW Peak Road. The proposed pipe would be approximately 2,300 ft along NW Eastview Drive, through an unnamed ravine, NW Eastview Drive, NW View Terrace Place and would involve a creek crossing.
- Alternative 2 – Pipe replacement / new pipe from the intersection of NW Eastview Drive to the intersection of NW Shoemaker Road and NW Peak Road. The proposed pipe would be approximately 2,000 ft along NW Eastview Drive and NW Shoemaker Rd and would involve a creek crossing.
- Alternative 3 – New developer pipe connection from PZ1 to PZ2. As part of this, the City would install an emergency BPS, with backup power, from the Low Zone to near Scappoose-Vernonia Highway and Blair Lane.

Upsizing a portion of Alternative 1 and Alternative 2 along NW Eastview Drive is proposed to address capacity issues, however it is recommended to implement the full Alternative 1 to also provide redundancy to the High Zone. For budgeting purposes, Alternative 1 was used as it is likely the highest cost alternative.

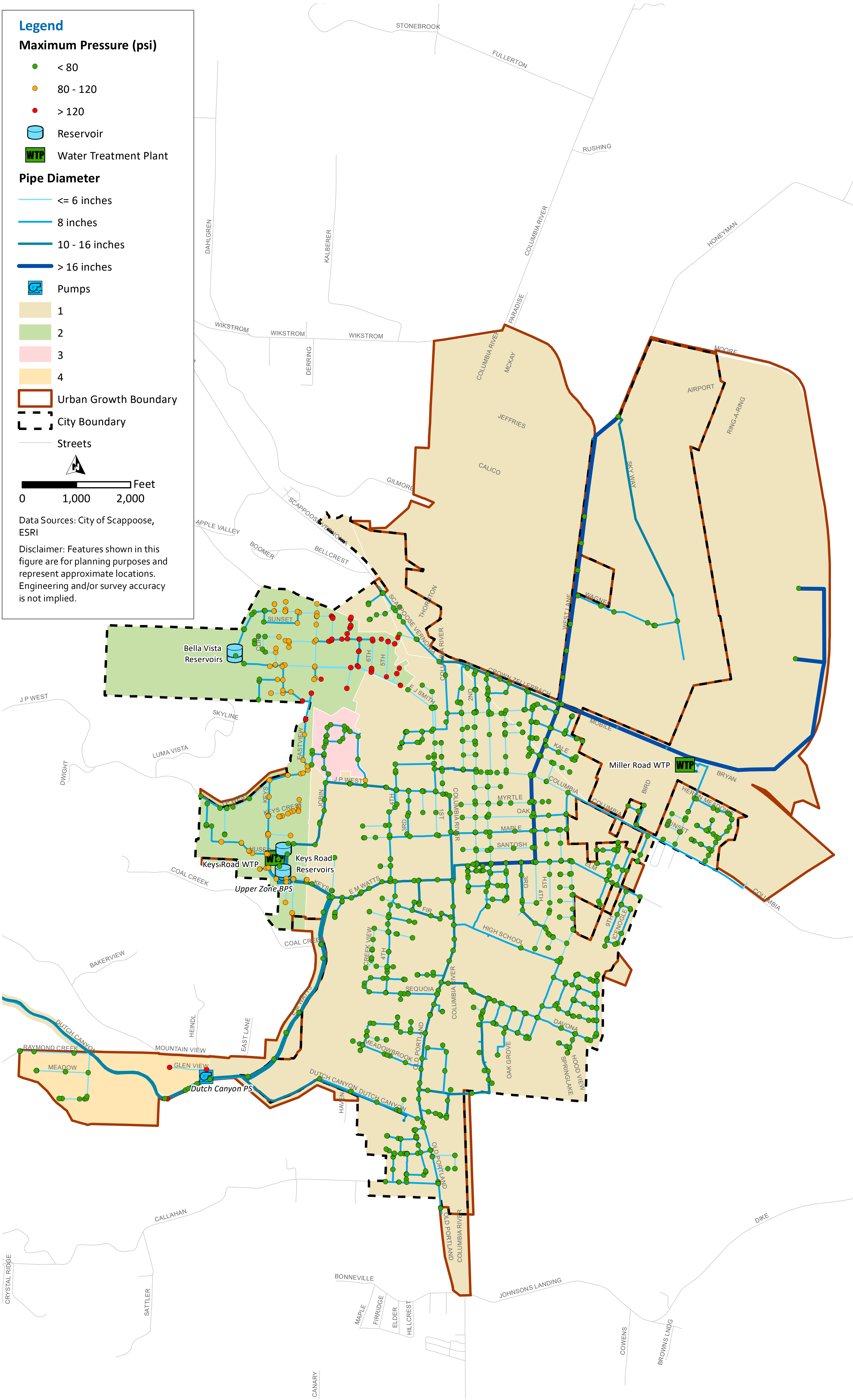


Figure 6.7 Short-Term Average Day Demand Maximum Pressures

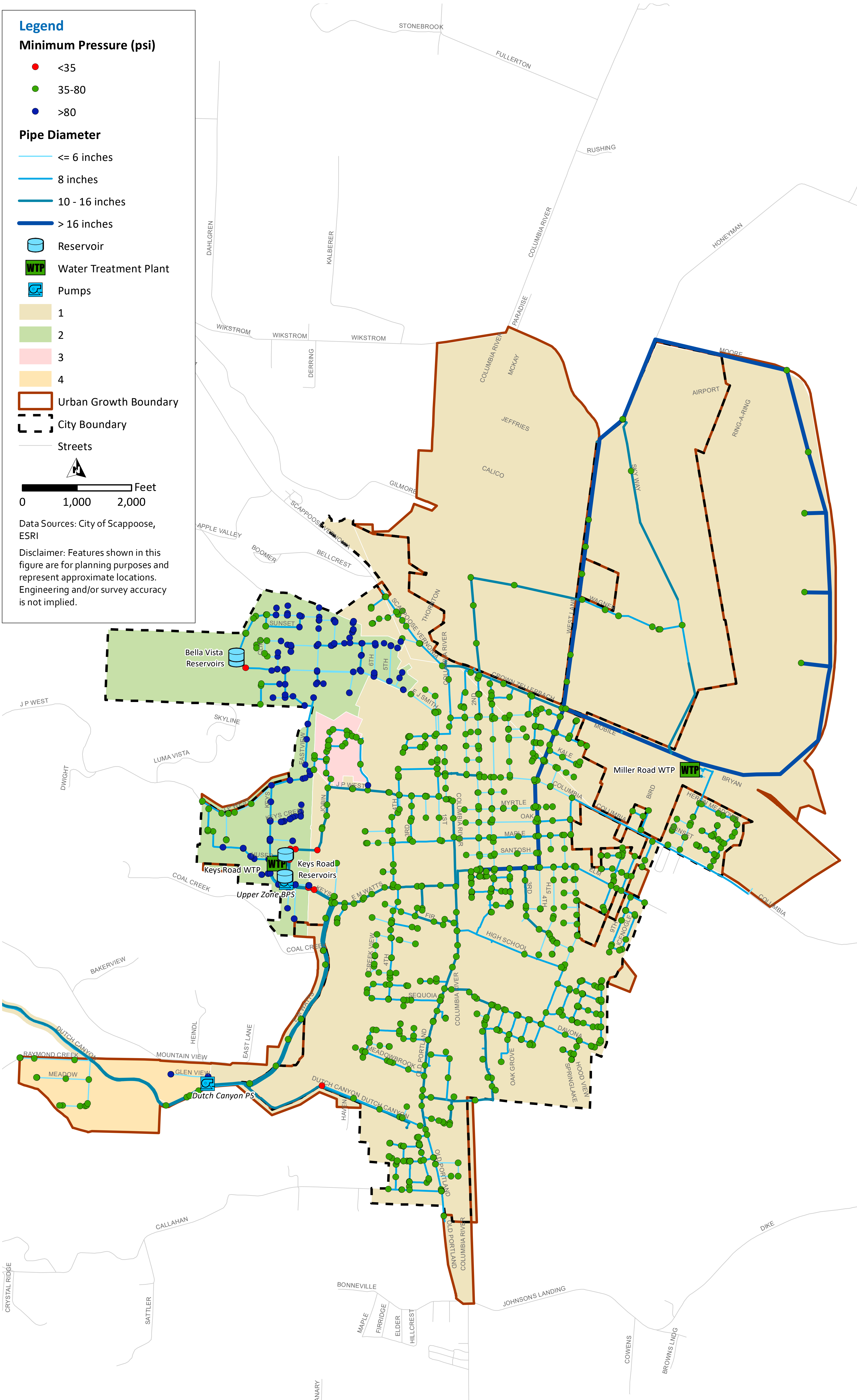


Figure 6.8 Long-Term Peak Hour Demand Pressures

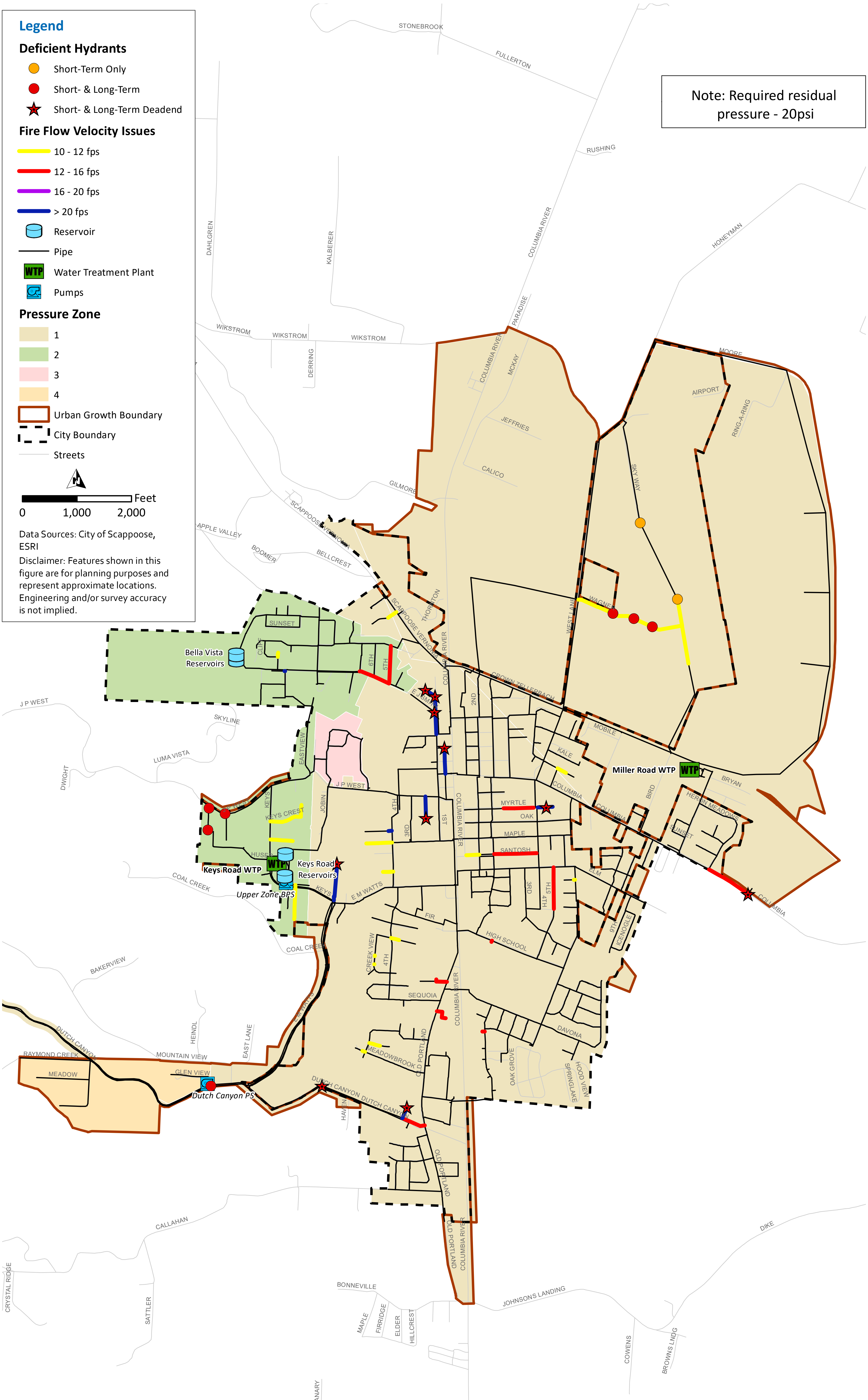


Figure 6.9 Areas Below Minimum Required Residual Pressure (20 psi) During MDD and Fire Flow

6.7.4 Capacity Improvements

Improvements have been recommended to resolve the deficiencies identified in the previous sections. Improvements include looping and pipe upsizing. The recommended improvements are shown in Figure 6.11. The available FF, with all recommended improvements, is shown in Figure 6.12 and detailed in Table 6.8. The columns used in Table 6.8 refer to the following:

- Model ID: Each pipe segment is assigned an ID. This is an alphanumeric number that starts with one letter indicating the type of project.
- Pressure Zone: Pressure zone the improvement is located in.
- Location: Street in which the improvement is proposed.
- Type of improvement: Pipe upsize, or new piping.
- Length: Estimated length of the proposed pipeline (in feet).
- Ex. Diam.: Diameter of the existing pipeline (in inches).
- New Diam.: Diameter of the proposed pipeline (in inches).
- Purpose: Reason for implementing each improvement (FF, pressure, other, etc.).

It is recommended the City address deficient dead-end and small diameter mains (high FF velocities) through an annual program. This allows the City to address these mains cost-effectively as part of nearby City projects or as new development occurs. A full breakdown of dead-end and small diameter mains is presented at the end of the Chapter.

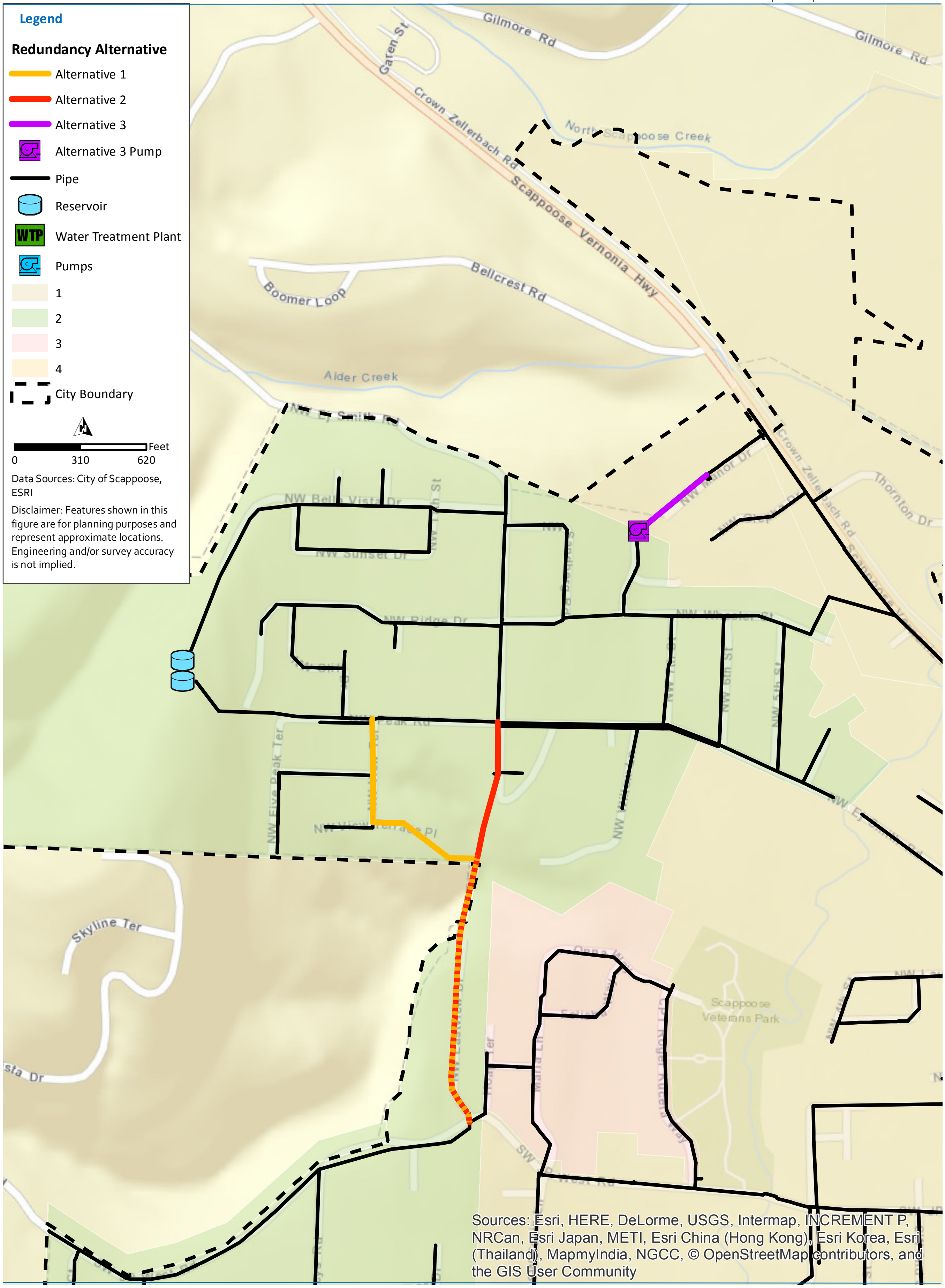


Figure 6.10 Overview of High Zone Redundancy Alternatives

Table 6.8 Overview of Proposed Improvements

Project ID	Model ID	Planning Period	Pressure Zone	Location	Type	Length	Ex. Diameter	New Diameter	Purpose
D-01	P2339	Short-Term	2	NW Eastview Drive	Replacement - WM1784	280	8	12	Fire Flow
	P2337	Short-Term	2	NW Eastview Drive	Replacement - WM1785	470	8	12	Fire Flow
	P2335	Short-Term	2	NW Eastview Drive	Replacement - WM1796	140	8	12	Fire Flow
	P2333	Short-Term	2	NW Eastview Drive	Replacement - WM1795	40	8	12	Fire Flow
	P2341	Short-Term	2	NW View Terrace	New	1,370	-	12	High Zone Redundancy
D-02	P2315	Short-Term	1	South of Meadowbrook	New	780	-	8	Looping/Fire Flow
D-03	P2283	Short-Term	1	Airport Annexation	New	1,560	-	12	Airport Loop
D-04	P2321	Long-Term	1	Dutch Canyon Road	New	1,580	-	12	Fire Flow
D-05	P2305	Long-Term	1	Airport Annexation	New	4,050	-	18	Looping New Development
D-06	P2303	Long-Term	1	Airport Annexation	New	1,780	-	12	Looping New Development

Figure 6.11 Overview of Recommended Improvements

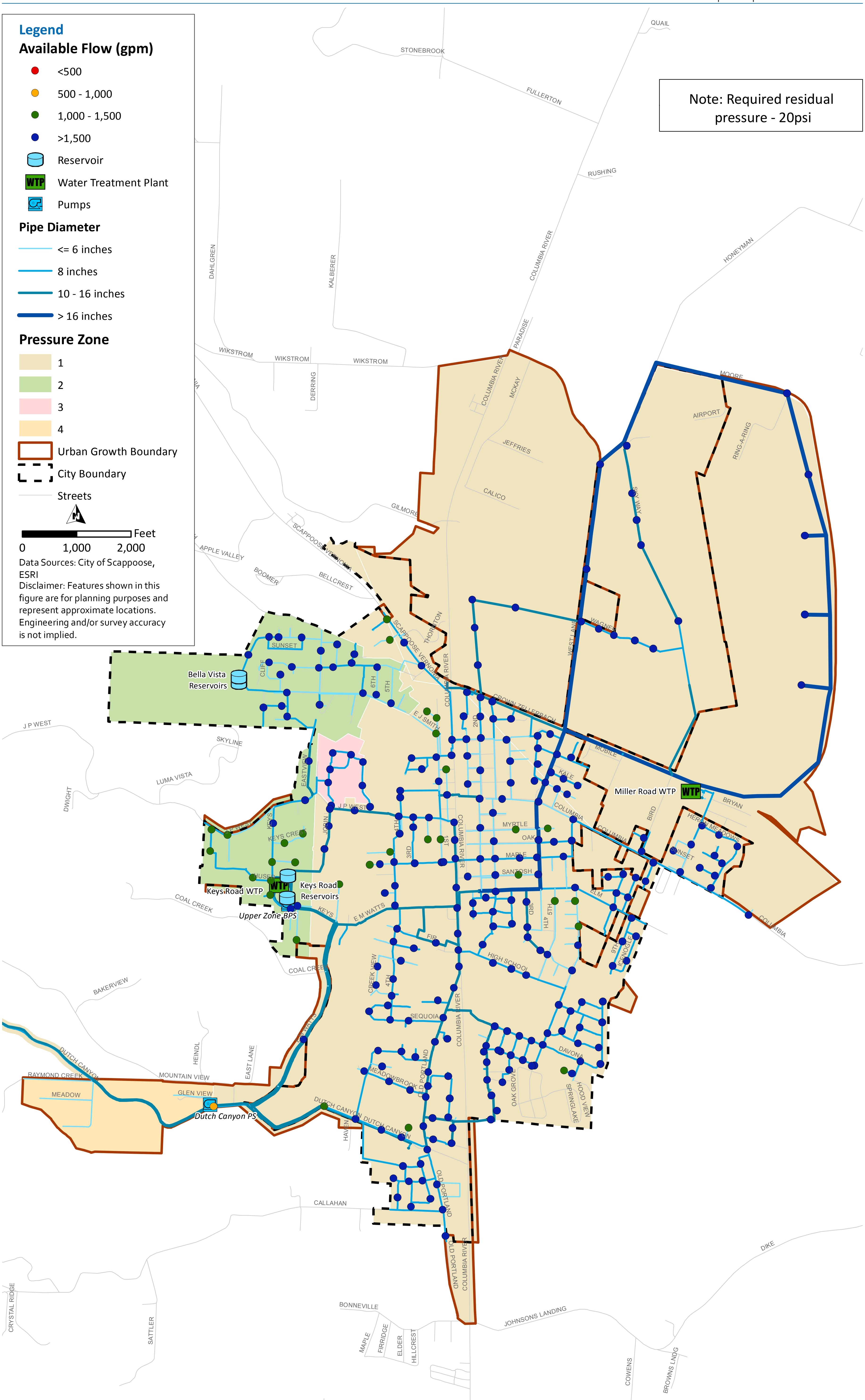


Figure 6.12 Improvement Scenario Fire Flow Results

6.8 Cost and Recommendations

This Chapter identified a number of improvements to address future system deficiencies. This section provides cost estimates for the recommended system improvements.

6.8.1 Cost Estimating Assumptions

Planning-level cost estimates were developed for the proposed system improvements. Costs provided are planning level estimates only and should be refined during pre-design of the projects. The cost estimates developed in this chapter are American Academy of Cost Engineers (AACE) Class 4 estimates. Class 4 estimates are budget level estimates. Actual costs may vary from these estimates by -30 percent to +50 percent.

All costs are in February 2018 dollars. No inflation rate is applied to the cost of these supply options. This allows project costs to be inflated as warranted in the future. The Engineering News-Record (ENR) U.S. 20-City Construction Cost Index for February 2018 is 10,889.

The cost estimates were based on construction costs inflated using cost factors shown in Table 6.9.

Table 6.9 Cost Factors

Cost Factor	Description	Factor
Contingency	Costs that may occur due to uncertainty in project scope and conditions.	30%
Planning/Engineering and City Admin	Cost for planning and design of project as well as City administration costs for completing the project.	25%

6.8.1.1 Booster Pump Stations

A new High Zone BPS was recommended as part of the pumping analysis. BPS construction costs were estimated using a unit construction cost based on the number of pumps and HP of the pumps. Table 6.10 provides the unit construction costs used. Unit construction costs include site work, pumps, a structure, all mechanical and electrical equipment, and a back-up generator.

Table 6.10 Pump Station Costs

Horsepower	Unit	Unit Construction Cost
0 to 199 HP	Per HP per Pump	$4,100
200 to 349 HP	Per HP per Pump	$3,300
350 to 649 HP	Per HP per Pump	$2,500
>650 HP	Per HP per Pump	$1,700

6.8.1.2 Storage Costs

The storage analysis recommended additional ground storage for the Low Zone. Unit construction costs for ground storage are presented in Table 6.11. Unit construction costs include site work, a structure, mechanical and electrical equipment, and piping to connect to the distribution system.

Table 6.11 Storage Costs

Element	Unit	Unit Construction Cost
Ground Storage	Gallon	$1.50

6.8.1.3 Distribution System Costs

Distribution system unit construction costs are presented in Table 6.12. These unit costs assume open-trench construction and include pavement cutting, excavation, hauling, shoring, pipe materials and installation, backfill material and installation, and pavement replacement.

Acquisition, easements, and right-of-way (ROW) may be required for some of the recommended projects. For the purpose of these cost estimates, pipeline corridors were assumed to be in public ROW, and do not require land acquisition.

Table 6.12 Distribution Costs

Element	Unit(1)	Unit Construction Cost
8 inch Pipe	LF	$180
10 inch Pipe	LF	$200
12 inch Pipe	LF	$220
16 inch Pipe	LF	$240
18 inch Pipe	LF	$260
24 inch Pipe	LF	$310

Note:
(1) LF: Linear Foot

6.8.2 Storage and Pumping Improvements

The pumping analysis recommended a new High Zone BPS with three 25 HP pumps. The total estimated project cost is $480,000.

The storage analysis recommended a new 2.0 MG reservoir located at Keys Road. The total estimated project cost is $4,650,000.

6.8.3 Capacity Improvements

Cost estimates for the distribution system capacity improvements detailed previously in Table 6.8 are shown in Table 6.13.

Table 6.13 Capacity Improvements Project Cost Estimates

Project ID	Location	Diameter (inches)	Length (ft.)	Total Project Cost
D-01	NW Eastview Drive / NW View Terrace	12	2,300	$790,000
D-02	South of Meadowbrook	8	780	$210,000
D-03	Airport Annexation	12	1,560	$530,000
D-04	Dutch Canyon Road	12	1,580	$540,000
D-05	Airport Annexation	18	4,050	$1,630,000
D-06	Airport Annexation	12	1,780	$610,000

6.8.4 Dead-End and Small Diameter Mains

A full summary of the dead-end and small diameter (high velocity) mains, including length and required diameter, is presented in Table 6.14. Were each of these to be addressed as individual projects, considering the unit construction costs presented in Table 6.12, the total cost is estimated to be $3,520,000. It is recommended the City address deficient dead-end and small diameter mains (high FF velocities) through an annual program. This would allow the City to address these mains more cost-effectively as part of nearby City projects or as new development occurs.

6.9 Environmental Impacts

The City does not anticipate environmental impacts from the proposed improvements. The City requires all projects to applicable meet local, state, and federal requirements. All water supply projects, discussed in Chapter 4, will be addressed through the water right and construction permit process. Distribution system improvements will mitigate environmental concerns, such as sediment control, per project permits.

Table 6.14 Capacity Improvements Project Cost Estimates

Pipe ID	Required FF	Pipe Length (ft)	Existing Diameter (in)	Upsized Diameter (in)	Pipe ID	Required FF	Pipe Length (ft)	Existing Diameter (in)	Upsized Diameter (in)
WM2078	2000	847	8	12	WM1223	1000	5	4	6
WM2400	2000	5	8	12	WM1154	1000	5	4	6
WM2398	2000	23	8	12	WM2464	2000	45	8	12
WM2184	2000	212	6	12	WM1037	1000	144	6	8
WM2183	2000	5	6	12	WM1035	1000	5	6	8
WM2180	2000	5	8	12	WM1028	1000	5	6	8
WM2179	2000	30	8	12	WM1011	1000	185	6	8
WM2177	2000	194	8	12	WM1001	1000	5	6	8
WM2176	2000	5	8	12	WM0995	1000	184	6	8
WM2151	2000	213	6	12	WM0993	1000	5	6	8
WM0493	2000	5	2	12	WM0928	1000	5	6	8
WM2150	2000	253	2	12	WM0908	1000	3	6	8
WM2626	2000	356	8	12	WM0900	1000	413	6	8
WM0494	2000	5	6	12	WM0878	1000	136	6	8
WM2066	2000	293	6	12	WM0877	1000	56	6	8
WM1546	2000	15	6	12	WM0876	1000	26	6	8
WM2032	2000	389	6	12	WM0875	1000	99	6	8
WM1547	2000	7	8	12	WM0874	1000	59	6	8
WM1548	2000	10	8	12	WM0873	1000	50	6	8
WM0031	2000	280	8	12	WM0872	1000	109	6	8
WM1899	2000	79	8	12	WM0871	1000	42	6	8
WM1955	2000	5	8	12	WM0870	1000	22	6	8
WM2000	1000	350	4	6	WM0869	1000	198	6	8
WM1999	1000	416	4	6	WM0693	1000	206	6	8
WM1998	1000	240	6	8	WM0690	1000	5	6	8
WM1962	1000	529	4	6	WM0683	1000	54	6	8
WM1961	1000	225	4	6	WM0684	1000	5	6	8
WM2002	1000	352	4	6	WM0501	1000	5	4	6
WM1166	1000	5	4	6	WM2067	1000	215	4	8
WM1931	1000	30	4	6	WM0350	3500	27	8	12
WM1928	1000	653	4	6	WM0064	3500	5	8	12
WM1927	1000	24	4	6	WM0342	3500	78	8	12
WM1924	1000	559	4	6	WM0062	3500	178	8	12
WM1922	1000	8	4	8	WM0338	3500	368	8	12
WM1916	1000	66	4	8	WM0349	3500	408	8	12
WM1905	1000	5	4	8	WM0059	3500	5	8	12

Table 6.14 Capacity Improvements Project Cost Estimates (continued)

Pipe ID	Required FF	Pipe Length (ft)	Existing Diameter (in)	Upsized Diameter (in)	Pipe ID	Required FF	Pipe Length (ft)	Existing Diameter (in)	Upsized Diameter (in)
WM1904	1000	12	4	8	WM0058	3500	488	8	12
WM1903	1000	72	4	8	WM0057	3500	5	8	12
WM1902	1000	608	4	8	WM0056	3500	691	8	12
WM2003	1000	160	4	8	WM0055	3500	5	8	12
WM1165	1000	5	4	8	WM0054	3500	5	8	12
WM2004	1000	115	4	8	WM0065	3500	36	8	12
WM1774	1000	628	6	8	WM0341	3500	317	8	12
WM1772	1000	5	6	8	WM0340	3500	145	8	12
WM2005	1000	202	4	6	WM0339	3500	155	8	12
WM1237	1000	5	4	6	WM0063	3500	25	8	12
WM2020	1000	395	4	8	WM0343	3500	8	8	12
WM2021	1000	5	4	8	WM0337	3500	53	8	12
WM2044	1000	177	6	8	WM0336	3500	54	8	12
WM1526	1000	25	6	8	WM0335	3500	6	8	12
WM1465	1000	75	6	8	WM0334	3500	6	8	12
WM1462	1000	5	6	8	WM0333	3500	126	8	12
WM1459	1000	256	6	8	WM0061	3500	5	8	12
WM1458	1000	22	6	8	WM0060	3500	17	8	12
WM1457	1000	192	6	8					

Chapter 7

OPERATIONS AND MAINTENANCE

7.1 Introduction

This chapter provides an evaluation of the City of Scappoose's (City's) operations and maintenance (O&M) requirements to effectively maintain their existing facilities and distribution system. O&M needs were identified from a condition assessment of the City's existing treatment facilities, discussions with City staff, and a pipeline remaining useful life (RUL) analysis.

7.2 Condition Assessment

A condition assessment of the City's treatment facilities was conducted and is detailed in Chapter 5 of this Water System Master Plan Update (Plan). The facilities were found to be well-maintained by City staff; however, the plants are aging and did not appear to meet current life and safety requirements in some cases. The condition assessment recommended:

- Life-safety audit.
- Repair and Replacement (R&R) program.
- Seismic resiliency upgrades.

To address these issues, it is recommended the City address life-safety issues immediately through a capital improvement project. R&R may be accomplished through a combination of capital projects and smaller projects funded through the O&M budget on an annual basis.

Seismic upgrades are recommended, especially when they can be implemented cost-effectively with other projects. State regulations provide for 50 years to implement a seismic mitigation plan; therefore, the City is not required to implement Seismic resiliency projects in the planning period.

7.3 Preventative Maintenance

O&M of facilities and distribution system were discussed with City Staff. City staff are able to operate the system well; however, there are not enough operations staff to meet the City's preventative maintenance goals. Preventative maintenance has been shown to reduce the overall cost of operating the water system by:

- Maintaining a high level of service in the distribution system and treatment plants,
- Increase equipment life and operating efficiency, and
- Cost effective maintenance by avoiding emergency repairs.

Staff indicated that an additional two operators would increase preventative maintenance.

7.4 Water Loss

The City's water losses, as shown in Chapter 3, have averaged almost 30 percent since 2011. For comparison purposes, water losses in the Pacific Northwest municipal utilities are recommended to be below 15 percent, with water losses commonly between 4 and 10 percent. Water loss includes both apparent and real losses. Apparent losses include water theft, meter inaccuracies, and data collection errors.

Real losses are physical losses from the distribution system including reservoir overflows, water main breaks, and water main leaks. If all 30 percent water loss was real losses, then it would equate to 83.9 million gallons per year of water.

Due to the potential revenue loss, we strongly recommend resolving water loss issues. Potential actions to reduce apparent losses include:

- <u>Conduct a Water Audit</u>. Conduct an industry standard water audit using the American Water Works Association (AWWA) methodology to identify sources of possible water loss. Chapter 3 of the Plan summarizes the City's past water production and consumption data using this methodology.
- <u>Calibrate source flow meters</u>. Calibrate of the City's six flow meters to reduce apparent losses that may be occurring due to inaccurate measurement.
- <u>Continue customer meter replacement</u>. Replace failing customer meters, which typically under-read flow as they age, to reduce apparent losses and provide accurate measurement for customer billing.

Water losses not addressed by these actions are real losses (leaks) in the distribution system, typically due to aging infrastructure. Leaks can occur due to corrosion of pipes, at joints and valves, and at fire hydrants. It is recommended that fire hydrants be inspected for leaks as part of annual testing, with special attention to poorly seated valves that are a common source of water leaks. Acoustic water loss testing is generally considered to be the most cost-effective method for identifying leaks within the distribution system. Due to the cost of testing, acoustic testing may be performed periodically (every 3 to 10 years). The City performed acoustic testing in 2016; therefore, it is recommended to prioritize other activities over water loss testing in the near-term.

Identified water leaks should be repaired as soon as possible. The decision to repair or replace a leaking pipeline should be considered on a case-by-case basis. Replacement should be considered for pipes with multiple leaks, small diameter, on a dead-end, or older pipes reaching the end of their useful life. The next section provides a desktop analysis of remaining useful life to aid in identifying these aging pipes.

7.5 Pipeline Repair and Replacement

Much of the City's core distribution system is made of steel or cast iron (CI) pipe that was constructed in the 1950s and 1960s. The surface water steel raw water main along Dutch Canyon Rd. is one of the oldest pipes and has had repeated leaks in recent years. Newer portions of the City's distribution system to the north and south have mainly been installed after 1990 with plastic pipe, with smaller amounts of ductile iron (DI) pipe.

Useful life is the length of time that a pipe is anticipated to remain in service. Useful life depends largely on pipe material, but can also depend on soil conditions, water constituents, and installation. Pipes at the end of their useful life generally have maintenance and repair costs that exceed the cost of replacement. Based on historical City O&M practices, end of useful life was defined as pipe failure, large leaks and/or structural failure.

Note, a useful life analysis does not predict when and where failures occur, rather it is a method to identify pipes for inclusion in an ongoing R&R program.

7.5.1 Pipe Material and Age

Pipe material and age are key data needed for an RUL analysis. Pipe material and installation dates were available from the City's geographic information system (GIS) records. Approximately 25 percent of the pipes in the City's GIS records did not have installation dates. For these pipes, assumptions were made based on the pipe age of similar pipe materials. To be conservative, pipes were assumed to have pipe ages similar to the oldest pipes of that material in the system. As the City continues to review and update its records, these assumptions can be better refined. The following summarizes the assumptions made to assign pipe ages:

- Steel pipe was assumed to have an install date of 1960.
- CI pipe was assumed to have an install date of 1970.
- DI pipe was assumed to have an install date of 1980.
- Plastic pipes, polyvinyl chloride (PVC), or C900, were assumed to have an install date of 1980.
- High-density polyethylene (HDPE) pipe was assumed to have an install date of 2000.
- Pipes in Pressure Zone, near the Bella Vista Reservoirs, were assumed to have an install date of 1970.

Pipe material for the City's raw water and distribution pipes is shown in Figure 7.1. Table 7.1 presents the total length of raw water pipe by installation decade and material type and Table 7.2 presents the total length of potable water pipe by installation decade and material type. Pipe age by installation decade is also shown in Figure 7.2.

Table 7.1 Raw Water Pipe Length by Decade Installed and Material Type

Material	Installation Decade						
	1950s	1960s	1970s	1980s	1990s	2000s	2010s
CI	5	4,497	0	0	0	0	0
DI	0	0	3,436	0	150	686	0
PVC	0	0	4,917	4,168	5,675	0	0
Steel	25,347	10	0	0	0	0	0
Unknown	0	2	597	0	0	27	0
Total by Decade	25,352	4,509	8,949	4,168	5,835	712	0

Table 7.2 Potable Water Distribution Pipe Length by Decade Installed and Material Type

Material	Installation Decade						
	1950s	1960s	1970s	1980s	1990s	2000s	2010s
CI	0	1,961	8,164	0	0	0	10
DI	0	0	12,870	1,913	2,565	1,922	1,735
GIP	0	0	56	1,314	0	0	14
HDPE	0	0	0	0	0	3,455	0
PVC	0	22	15,608	26,342	40,740	41,171	23,861
Steel	4,208	31,765	4,489	10	258	0	26
Unknown	0	0	714	396	512	94	5
Total by Decade	4,208	33,749	41,900	29,975	44,076	46,641	25,651

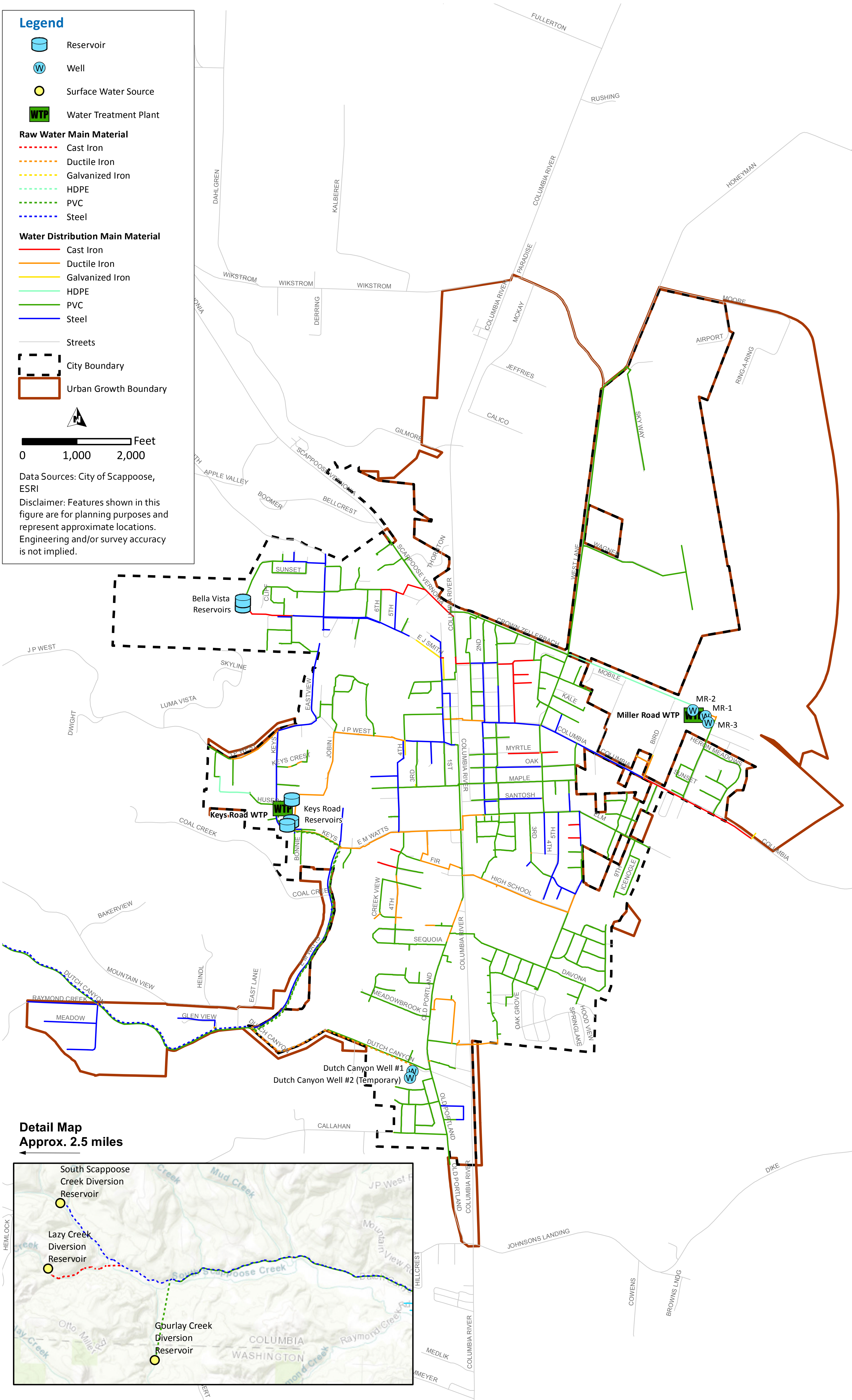
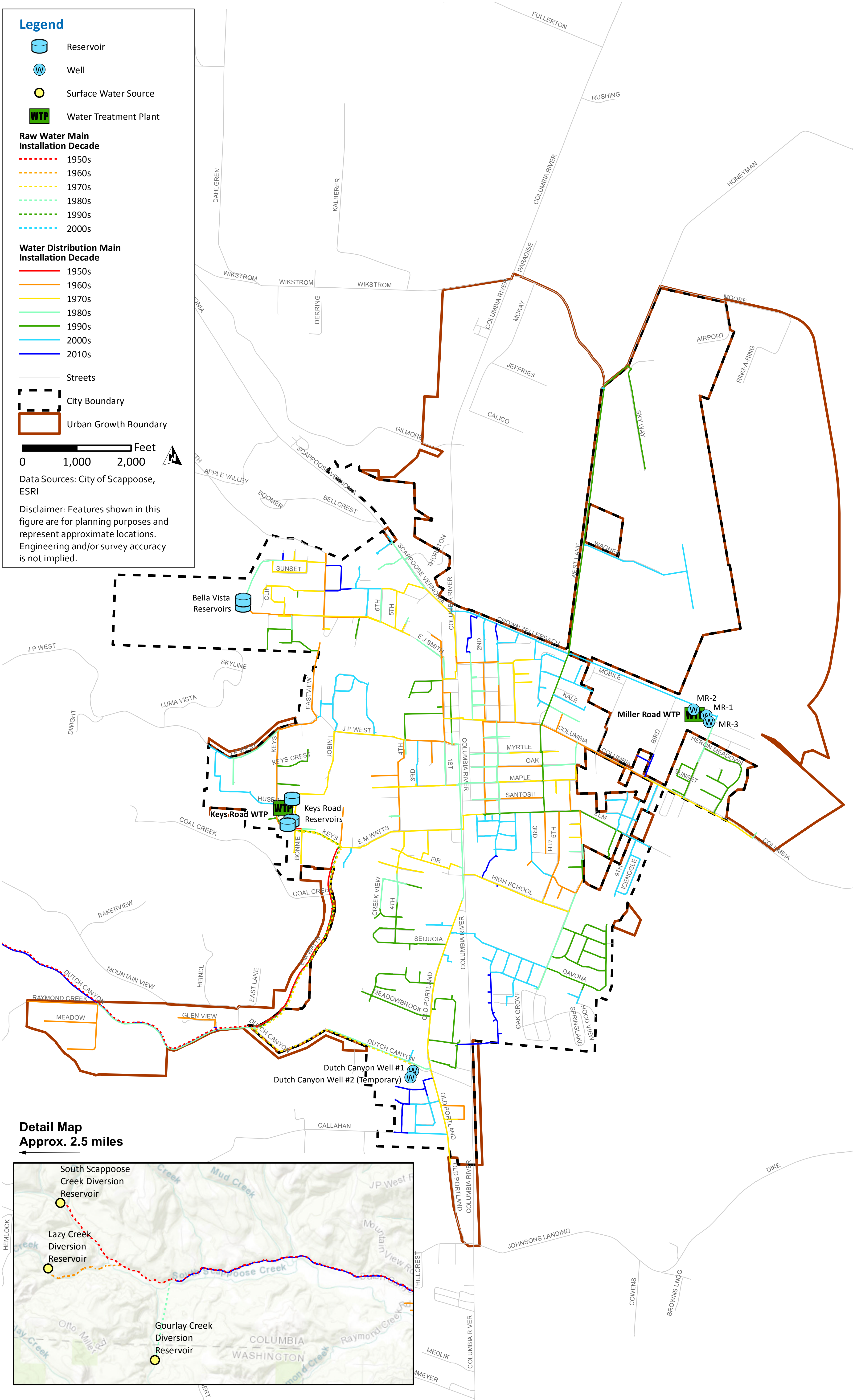


Figure 7.1 Water Mains by Material


Legend
Reservoir
Well
Surface Water Source
Water Treatment Plant
Raw Water Main Installation Decade
1950s
1960s
1970s
1980s
1990s
2000s
Water Distribution Main Installation Decade
1950s
1960s
1970s
1980s
1990s
2000s
2010s
Streets
City Boundary
Urban Growth Boundary
Feet
0 1,000 2,000
Data Sources: City of Scappoose, ESRI
Disclaimer: Features shown in this figure are for planning purposes and represent approximate locations. Engineering and/or survey accuracy is not implied.
Bella Vista Reservoirs
Keys Road WTP
Keys Road Reservoirs
Miller Road WTP
MR-1
MR-2
MR-3
Dutch Canyon Well #1
Dutch Canyon Well #2 (Temporary)
Detail Map
Approx. 2.5 miles
South Scappoose Creek Diversion Reservoir
Lazy Creek Diversion Reservoir
Gourlay Creek Diversion Reservoir

7.5.2 Useful Life

Useful life of a given pipe material combined with the age of the pipe provides the basis for RUL calculations. The estimated useful life of each pipe material was based on industry standards and is presented in Table 7.3.

Nearly 60 percent of the City's pipes are PVC pipes, designated as C900, PVC, or CL150 in the City's GIS records. The City has PVC pipes installed as early as the 1970's, when the AWWA first approved AWWA C900 standards for PVC pipe for water distribution. Modern plastics have advanced, with useful life up to 100 years. However, a 75 year useful life, within industry ranges, was assumed to account for variety in types of PVC pipe in the system.

The other dominant material in the City's system is steel pipe, installed mainly in the 1950s and 1960s. Steel pipe was assumed to have a useful life of 70 years. The steel pipes in the City's system are some of the oldest in the system. Frequent water leaks detected in the system are a strong indication that these pipes may be reaching the end of their useful life.

CI pipes were assumed to have a useful life of 75 years and DI pipes were assumed to have a useful life of 85 years. There are a small percentage of galvanized iron pipes (GIP) in the system, which was assumed to have a useful life of 50 years. HDPE pipe has been installed within the last fifteen years and was assumed to have a useful life of 100 years. Pipes with no known material type were assumed to have a useful life of 50 years.

It is important to note that actual useful life of an individual pipe can vary widely due to soil, groundwater, and installation conditions. It is recommended that the condition of pipes being replaced be noted for consideration in future analyses, which will increase the accuracy of the useful life estimates.

Table 7.3 Useful Life of Pipes

Material	Useful Life (years)
CI	75
DI	85
GIP	50
HDPE	100
PVC	75
Steel	70
Unknown	50

7.5.3 Remaining Useful Life Analysis

RUL is defined as the length of time left before a pipe's maintenance costs will likely exceed its cost of replacement. Pipe age, material type, and chosen useful life for each pipe material were used to determine the RUL of the City's pipes. The useful life of each pipe was calculated based on the installation year and the material-specific useful life. The period of time between the end of the useful life and today is the RUL. A PVC pipe installed in 2000 has a useful life of 75 years and will reach the end of its useful life in 2075. From the current year, 2018, this PVC pipe's RUL would be 57 years.

The RUL of pipes by replacement decade is shown throughout the system in Figure 7.3. The vast majority of the City's raw water and potable water pipes, 75 percent have useful life exceeding 20 years, outside the planning period. Pipe replacement will occur throughout the central, older portion of the City and out all the way to the City's surface water diversion structures. Pipes exceeding their useful life will continue to provide service, but pose a greater risk for pipe breaks and leakage.

7.5.3.1 Raw Water Mains

The RUL of raw water mains are summarized by material in Table 7.4. The City's oldest piping, the steel raw water mains from the surface water diversion structures, has reached the end of its useful life. Approximately 26,000 linear feet, or 52 percent, of City's raw water mains are recommended to be replaced in the 2010s and 2020s. This encompasses all of the City's steel raw water pipe.

Table 7.4 Raw Water Pipe Remaining Useful Life

Material	RUL <=10 Years (feet)	10 < RUL <=20 Years (feet)	RUL > 20 Years (feet)
CI	0	5	4,497
DI	0	0	4,272
GIP	0	0	0
HDPE	0	0	0
PVC	0	0	14,760
Steel	25,347	10	0
Unknown	599	0	27

7.5.3.2 Potable Water Mains

The replacement time period for potable water pipes is presented in Table 7.5. In addition to replacing steel raw water pipe, the steel pipe throughout the distribution system, a portion of the steel potable water pipe, is also expected to reach its useful life within the next 10 years. The vast majority of the steel potable water pipe is expected to reach its useful life in the next 20 years. Approximately 36,000 linear feet, or 88 percent of the City's steel water distribution pipes are recommended to be replaced in the 2020s and 2030s. There is a small amount, 1,300 linear feet, of GIP that should be replaced within the next 20 years as well.

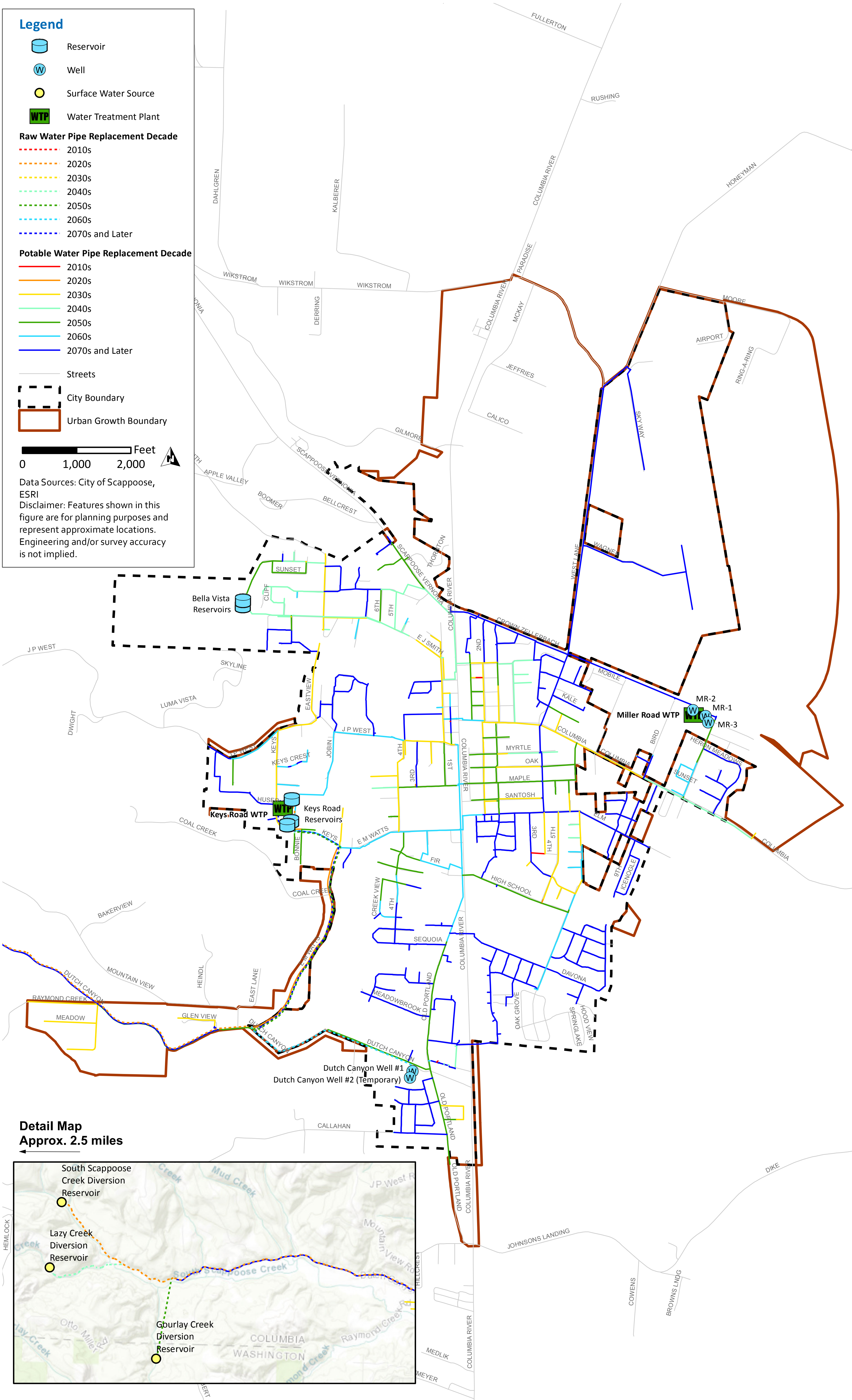


Figure 7.3 Remaining Useful Life

Table 7.5 Potable Water Pipe Remaining Useful Life

Material	RUL <=10 Years (feet)	10 < RUL <=20 Years (feet)	RUL > 20 Years (feet)
CI	0	0	10,135
DI	0	0	21,005
GIP	56	1,314	14
HDPE	0	0	3,455
PVC	0	18	147,727
Steel	4,208	31,765	4,784
Unknown	1,499	396	611
Total	**5,764**	**33,493**	**187,730**

7.6 Water Main Repair and Replacement Program

It is recommended that the City maintain an annual water main R&R program to address localized issues or to participate in joint projects that effectively replace aging pipe. When considering a pipe R&R program, short-term and long-term planning periods were considered based on RUL of 10 years or less, 10 to 20 years, and greater than twenty years.

The length of pipe reaching its useful life each year throughout the planning period is shown in Figure 7.4. Pipe replacement throughout the planning period is dominated by replacement of the steel raw and potable water pipes. The steel raw water pipes from the surface water diversion structures, installed in 1955, will reach the end of their useful life in 2025. It is important to note that this pipe would not need to be replaced should the City choose to abandon their surface water supplies in the future.

The majority of the City's steel potable water pipes were assumed to be installed in 1960, and reach the end of their useful life in 2030. The City will need to replace approximately 2,100 linear feet per year to address all pipes that reach the end of their useful life within the planning period.

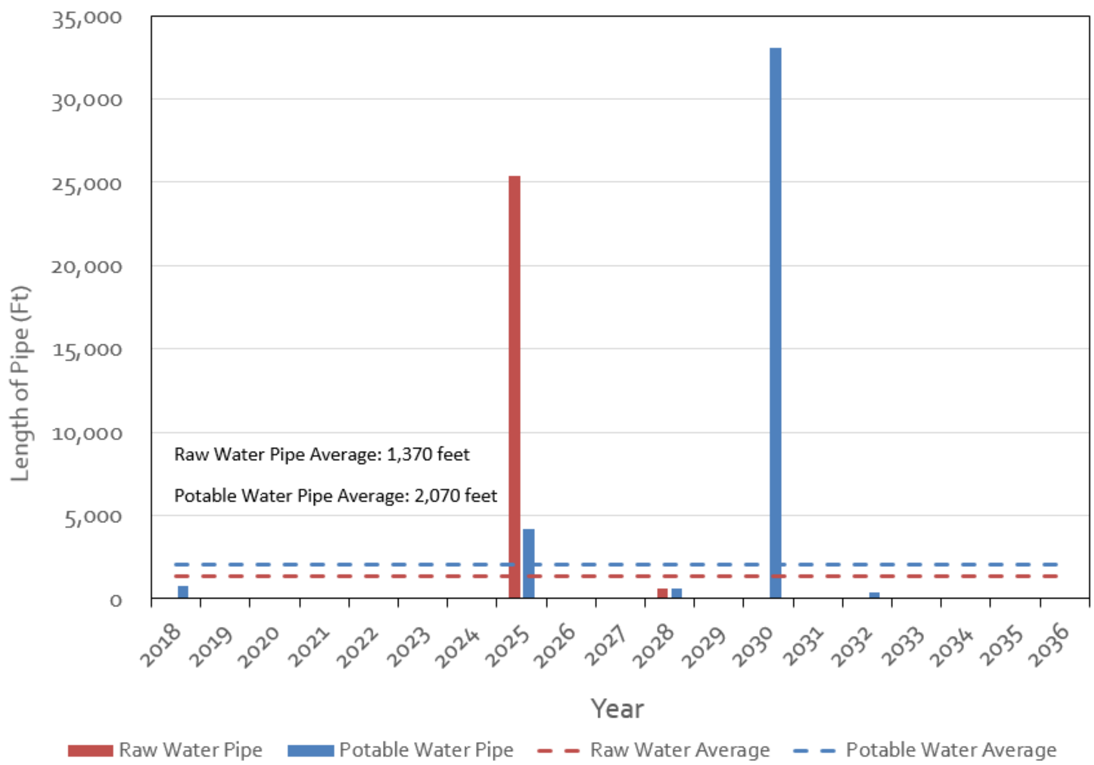


Figure 7.4 Length of Pipe Replacement by Year (2018-2036)

7.6.1 R&R Program Costs

This section provides cost estimates for the City's water main R&R program. An annual cost for replacement over the next 20 years is presented to aid in establishing a Pipe R&R program.

7.6.1.1 Cost Estimating Assumptions

Planning-level cost estimates were developed for the R&R program. Costs provided are planning level estimates only and should be refined during pre-design of the projects. The cost estimates developed in this chapter are American Academy of Cost Engineers (AACE) Class 4 estimates. Class 4 estimates are budget level estimates. Actual costs may vary from these estimates by -30 percent to +50 percent.

All costs are in February 2018 dollars. No inflation rate is applied to the cost of these supply options. This allows project costs to be inflated as warranted in the future. The Engineering News-Record (ENR) U.S. 20-City Construction Cost Index for February 2018 is 10,889.

The cost estimates were based on construction costs inflated using cost factors shown in Table 7.6.

Table 7.6 Cost Factors

Cost Factor	Description	Factor
Contingency	Costs that may occur due to uncertainty in project scope and conditions.	30%
Planning/Engineering and City Admin	Cost for planning and design of project as well as City administration costs for completing the project.	25%

7.6.1.2 Pipeline Unit Costs

Distribution system unit construction costs are presented in Table 7.7. These unit costs assume open-trench construction and include pavement cutting, excavation, hauling, shoring, pipe materials and installation, backfill material and installation, and pavement replacement. In addition to the unit costs in Table 7.7, pipeline costs were assumed to include 25 percent engineering, legal, and administration costs and a 30 percent contingency.

Acquisition, easements, and right-of-way (ROW) may be required for some of the recommended projects. For the purpose of these cost estimates, pipeline corridors were assumed to be in public ROW, and do not require land acquisition.

New pipes were assumed to have the same diameter as existing pipe, except for pipes smaller than 8 inches. Pipes with diameters less than 8 inches were assumed to be replaced with 8-inch diameter pipe.

Table 7.7 Distribution Costs

Element	Unit(1)	Unit Construction Cost
8 inch Pipe	LF	$180
10 inch Pipe	LF	$200
12 inch Pipe	LF	$220
16 inch Pipe	LF	$240
18 inch Pipe	LF	$260
24 inch Pipe	LF	$310

Note:
(1) LF: Linear Foot.

7.6.1.3 Water Main R&R Program Cost

The total cost for the water main R&R program was determined using the above assumptions and the replacement timing defined from the RUL analysis. The total cost for program to address the remaining potable pipe R&R within the planning period is $9.9 million. These costs do not include small diameter and dead-end mains identified in Chapter 6 to be upsized or looped. Costs for these replacements were assumed to be covered by the small diameter and dead-end main replacement program. If completed as part of the R&R program, these mains would add a total cost estimated to be $3.5 million. The combined program would have an annual budget of:

- $520,000 per year for R&R Program.
- $176,000 per year for Small Main and Dead-end replacement program.
- Combined program budget of approximately $700,000 per year.

These costs do not include the costs to replace the steel raw water pipes that were already evaluated in Chapter 4 as part of the supply alternatives analysis, at a total cost of $7.7 million. This replacement cost may not be necessary should the City pursue a future supply alternative that does not continue use of the existing surface water system.

The City should work to identify ways to combine pipe R&R and small diameter main projects when possible, to more cost-effectively complete these projects. In addition, the City may be able to more cost-effectively address pipe replacement and upsize projects by considering geographically concentrated projects that address multiple concerns and incorporate other utilities, such as sewer main R&R and roadway resurfacing.

7.7 Summary

The City's water system is generally well maintained, but is aging. The O&M evaluation evaluated treatment facilities and the distribution system. The following projects and programs are recommended to address O&M identified issues:

- Conduct life-safety audit of the treatment plants and address deficiencies.
- Conduct a water audit to identify causes of the City's high water loss rate. Short-term actions will likely include calibration of source water meters and continued replacement of failing customer meters.
- Consider the addition of two water operators to aid in preventative maintenance activities.
- Create an annual pipeline R&R program. It is recommended the program be funded at approximately $500,000 per year to replace steel pipes in the distribution system reaching the end of their usable life in the next 20 years.

Chapter 8

CAPITAL IMPROVEMENT PLAN

8.1 Introduction

This chapter summarizes the City of Scappoose's (City's) comprehensive capital improvement plan (CIP) for the water system that is based on the analyses presented in previous Chapters. The purpose of the CIP is to provide the City with a guideline for planning and budgeting of its water system. The CIP consists of schedule and cost estimates in present dollars (March 2018) for each project. The total project costs are presented in each Chapter; this Chapter integrates the projects, applies developer funding, and provides project timing.

8.1.1 Capital Project Categories

Capital projects were categorized by the nature of the infrastructure:

- Supply (S).
- Treatment (T).
- Distribution (D).
- Pump Stations (PS).
- Storage (ST).
- Miscellaneous (Misc).

Projects were divided into three categories for funding:

- Capacity: Expand infrastructure to accommodate future demands.
- Repair and Replacement (R&R): Repair and replacement of aging infrastructure.
- Upgrades: Projects that increase the level of service from existing infrastructure (i.e. reliability, seismic mitigation, etc.).

The City has entered into agreements with developers to fund system infrastructure required for new development. Developer funding, presented as a percentage of total project costs, was accounted for in the CIP summary. Total project costs are presented in the project "cut sheets" to aid in City planning and then discounted based on developer funding.

8.1.2 Capital Planning Periods

CIP projects were allocated into one of two planning periods referenced in previous chapters:

- Short-term (2018-2028)
- Long-term (2029-2038)

Project timing was developed from the technical analyses considering financial and staff resources. Projects within the short-term planning horizon were allocated to individual years. Projects in long-term planning horizons do not provide the same level of specificity, reflecting the uncertainty in future needs and City resources. The project timing in this Chapter is subject to change, as the City regularly reviews and updates its CIP based on changing conditions and priorities.

8.2 Cost Estimating Assumptions

The project cost assumptions are presented in each Chapter. As previously discussed, the CIP cost estimates presented in this chapter are American Academy of Cost Engineers (AACE) Class 4 estimates. Class 4 estimates are budget level estimates. Actual costs may vary from these estimates by -30 percent to +50 percent. These costs were determined based on the City's and Carollo Engineers, Inc., (Carollo's) perception of current conditions at the project locations.

All costs are in March 2018 dollars. No inflation rate is applied to the cost of future projects. This allows project costs to be inflated as warranted in the future. The Engineering News-Record (ENR) U.S. 20-City Construction Cost Index for March 2018 is 10,889. The estimates are subject to change as the project design matures. Cost of labor, materials, and equipment may vary in the future.

8.3 Capital Improvement Projects

8.3.1 Supply Projects

As presented in Chapter 4, development of new supplies is required to meet growing system demand. These included development of new groundwater wells in the short-term with long-term supply sources to be determined. Potential well locations are shown in Figure 8.1. Details on each supply project are provided in Chapter 4 and summarized below.

8.3.1.1 Dutch Canyon Well #2 (S-01)

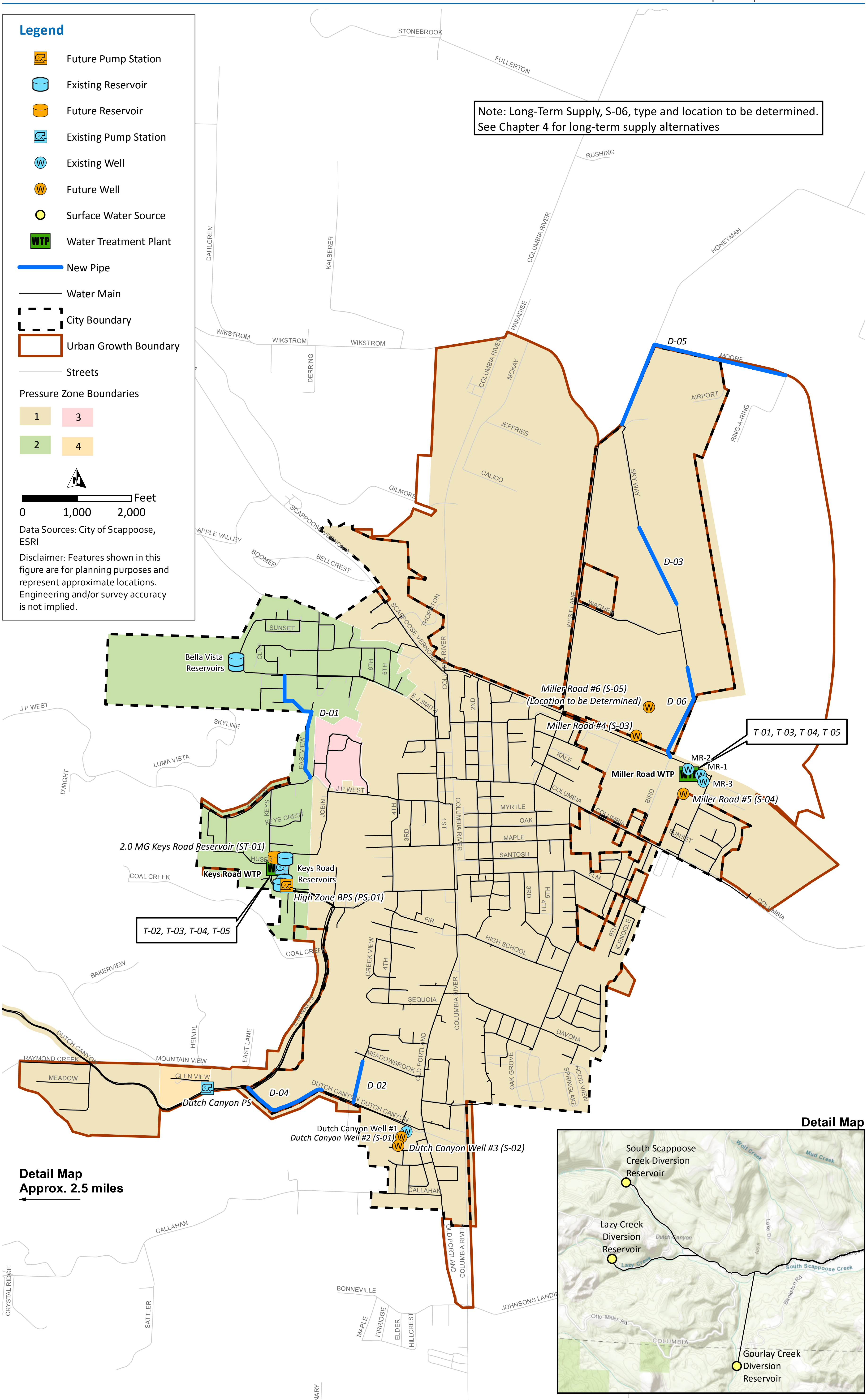
Complete construction of Dutch Canyon Well #2, including mechanical, electrical, instrumentation, and controls. Project costs were based on the Dutch Canyon Replacement Well Pump Station Improvements Preliminary Design Memo (Carollo 2017). This project is estimated to cost $480,000.

8.3.1.2 Dutch Canyon Well #3 (S-02)

Develop Dutch Canyon Well #3 near the City's existing Dutch Canyon wells. It is assumed the City will acquire new property (assumed 1/3 acres) to site the well where it will not interfere with the yield of the City's existing wells. Well development costs include development of the production well, well pump and appurtenances, well house, civil site improvements, and instrumentation and control. Included with this project are costs for 1,500 linear feet (LF) of raw water transmission main to connect the raw water main from the existing wells. Raw water transmission main construction includes open-trench construction and includes pavement cutting, excavation, hauling, shoring, pipe materials and installation, backfill material and installation, and pavement replacement. The total projected cost for this project is $2,100,000, to be phased over two years with design occurring in year one and construction occurring in year two.

8.3.1.3 Miller Road Well #4 (S-03)

Develop a new well in the vicinity of the City's existing Miller Road treatment plant. The new well will be developed in the E. Airport development per the development agreement. It is assumed this well will be developed at the CZ-1 site set forth in the Application for Permit Amendment on filed on February 24, 2016 for Permit # G-15491. Project costs represent costs for City administration based on 10 percent of the estimated total project cost. No land costs were included for the project.


Legend
Future Pump Station
Existing Reservoir
Future Reservoir
Existing Pump Station
Existing Well
Future Well
Surface Water Source
Water Treatment Plant
New Pipe
Water Main
City Boundary
Urban Growth Boundary
Streets
Pressure Zone Boundaries
1
3
2
4
Feet
0 1,000 2,000
Data Sources: City of Scappoose, ESRI
Disclaimer: Features shown in this figure are for planning purposes and represent approximate locations. Engineering and/or survey accuracy is not implied.
Note: Long-Term Supply, S-06, type and location to be determined.
See Chapter 4 for long-term supply alternatives
D-05
D-03
D-06
D-01
D-02
D-04
Miller Road #6 (S-05)
(Location to be Determined)
Miller Road #4 (S-03)
T-01, T-03, T-04, T-05
MR-2
MR-1
MR-3
Miller Road WTP
Miller Road #5 (S-04)
Bella Vista Reservoirs
2.0 MG Keys Road Reservoir (ST-01)
Keys Road WTP
Keys Road Reservoirs
High Zone BPS (PS-01)
T-02, T-03, T-04, T-05
Dutch Canyon PS
Dutch Canyon Well #1
Dutch Canyon Well #2 (S-01)
Dutch Canyon Well #3 (S-02)
Detail Map
Approx. 2.5 miles
Detail Map
South Scappoose Creek Diversion Reservoir
Lazy Creek Diversion Reservoir
Gourlay Creek Diversion Reservoir

8.3.1.4 Miller Road Well #5 (S-04)

Develop a new well in the vicinity of the City's existing Miller Road treatment plant. The new well is assumed to be located within Miller Park and will be treated at the Miller Road water treatment plant (WTP). Well development costs include development of the production well, well pump and appurtenances, well house, civil site improvements, and instrumentation and control. Included with this project are costs for 1,500 LF of raw water transmission main to pump to the Miller Road WTP. Raw water transmission main construction includes open-trench construction and includes pavement cutting, excavation, hauling, shoring, pipe materials and installation, backfill material and installation, and pavement replacement. The project includes the purchase of 0.33 acres of additional land to mitigate the potential loss of park land. The total projected cost for this project is $1,970,000, to be phased over two years with design occurring in year one and construction occurring in year two.

8.3.1.5 Miller Road Well #6 (S-05)

Develop a new well in the vicinity of the City's existing Miller Road treatment plant. The location of the new well has not been identified and may result in higher transmission and land costs. Well development costs include development of the production well, well pump and appurtenances, well house, civil site improvements, and instrumentation and control. Included with this project are costs for 1,500 LF of raw water transmission main to pump to the Miller Road WTP. Raw water transmission main construction includes open-trench construction and includes pavement cutting, excavation, hauling, shoring, pipe materials and installation, backfill material and installation, and pavement replacement. The project includes the purchase of 0.33 acres of land for the well site. The total projected cost for this project is $2,100,000, to be phased over two years with design occurring in year one and construction occurring in year two.

8.3.1.6 Long-Term Supply (S-06)

Four alternatives for long-term supplies were evaluated; however, there are unknowns in all alternatives that may cause large changes in costs. Action items to confirm supply effectiveness and refine costs of the supply options are presented in Chapter 4. As the City completes the activities, they should revise the yield, water quality, and costs assumptions of this Chapter.

A budget placeholder for long-term supply of $12,650,000 was recommended as the City completes these activities, which is based on expected costs for either additional Miller Road Wells or a new Ranney well source, which are detailed in Chapter 4.

8.3.2 Treatment Projects

As presented in Chapter 5, Treatment projects include repair and replacement improvements and additional studies to maintain existing supplies and treatment infrastructure. Carollo recommends these projects be completed in the short-term to meet the needs of projected growth. Details on each treatment project were provided in Chapter 5; a high level summary of the key recommendations are provided below.

8.3.2.1 Miller Road R&R (T-01)

Repair and replacement improvement projects consist of life safety, rehab, and seismic upgrades to existing Miller Road WTP facilities. These projects include procuring critical spare parts, injectors, metering pumps, and control valves, supervisory control and data acquisition (SCADA) refinements, incorporating spill containment, building a new backwash basin, and installing an oxidation/reduction potentiometer (ORP) sensor. The total projected cost for this project is $650,000. All R&R projects are planned to be completed in the short-term planning period. These costs are based on Carollo's condition assessment and estimate for anticipated recommendations of future studies. These costs are subject to change based on these future recommendations.

8.3.2.2 Keys Road R&R (T-02)

Repair and replacement improvement projects consist of nonstructural rehab to existing Keys Road WTP facilities. These projects include procuring critical spare parts, injectors, metering pumps, and control valves, incorporating spill containment, covering greensand filter, and installing ORP sensor.

The total projected cost for this project is $340,000. These costs are based on Carollo's condition assessment and estimate for anticipated recommendations of future studies. These costs are subject to change based on these future recommendations.

8.3.2.3 Supply and Treatment Plant Level of Service Goals (T-03)

Develop Level of Service (LOS) goals for the performance of existing facilities to improve communication and balance the City staff's everyday operations and problem solving. These goals are long-term supply alternatives that aim to represent the City's overall water system goals. The placeholder budget cost for this study is $20,000 and should be updated after the study.

8.3.2.4 Seismic and Life-Safety Audit (T-04)

Perform seismic and life-safety audit program on all Miller Road treatment facilities. The audit will identify potential seismic performance deficiencies and potential life-safety deficiencies in the structural connections, equipment anchors, mechanical and electrical systems, and other ancillary components. The placeholder budget cost for this audit is $160,000 and should be updated after the audit. Costs include anticipated costs associated with this audit, as well as a placeholder for the costs associated with implementation of the recommendations from this audit.

8.3.2.5 Treatment Capacity and Operations Optimization Study (T-05)

Conduct treatment capacity and operations optimization study on both surface water and groundwater. This study will evaluate the overall pre-treatment and filter performance to identify and provide opportunities to maximize the value of the City's existing treatment infrastructure while mitigating potential treatment challenges. The placeholder budget cost for this study is $290,000 and should be updated after the study. Costs include anticipated costs associated with this study, as well as a placeholder for the costs associated with implementation of the recommendations from this study.

8.3.3 Distribution Projects

As presented in Chapter 6, improvements to the distribution system in response to development are needed to address capacity limitations with existing infrastructure and for increase operation and maintenance (O&M) flexibility. The locations of distribution system improvement projects are shown in Figure 8.1. Figure 8.2 shows small diameter and dead end mains identified in Chapter 6 as well as pipes reaching the end of their useful life as identified in Chapter 7. Details on projects are provided in Chapter 6 and summarized below.

8.3.3.1 NW Eastview Drive/NW View Terrace (D-01)

This project consists of replacing existing 8-inch main along NW Eastview Dr. to the intersection of NW Eastview Terrace and NW Peak Rd. The pipe would run along NW Eastview Dr., through an unnamed ravine to NW Eastview Terrace then along NW Eastview Dr., and would involve a creek crossing. This project increases the hydraulic capacity between Bella Vista Reservoirs to the southern portion of the pressure zone to address fire flow limitations off SW JP W Rd. In addition, this is an aging pipeline and pipeline replacement will add additional capacity. Costs assume open-trench construction and include pavement cutting, excavation, hauling, shoring, pipe materials and installation, backfill material and installation, and pavement replacement. Construction costs do not account for wetland or stream mitigation or geotechnical stabilization for steep slopes or landslide prone areas. The total projected cost for this project is $790,000 and is expected to be phased over two years, with design occurring in year one and construction occurring in year two.

8.3.3.2 SW 5th Street Connection (D-02)

Construct a new 8-inch pipe on the future SW 5th St. (not currently platted) between SW Dutch Canyon Rd. and Havlik Dr. Costs assume open-trench construction and include pavement cutting, excavation, hauling, shoring, pipe materials and installation, backfill material and installation, and pavement replacement. The project is anticipated to be included in the future road right-of-way and does not include land acquisition. The total projected cost for this project is $210,000 and is expected to be phased over two years, with design occurring in year one and construction occurring in year two.

8.3.3.3 Sky Way Drive Connection to Airport Annex (D-03)

Connect the existing water main on Skyway Dr. to the 8-inch tee from Wagner Ct. using 12-inch piping. This project is assumed to be developer-funded and completed. A budget of $50,000 was included for City administration.

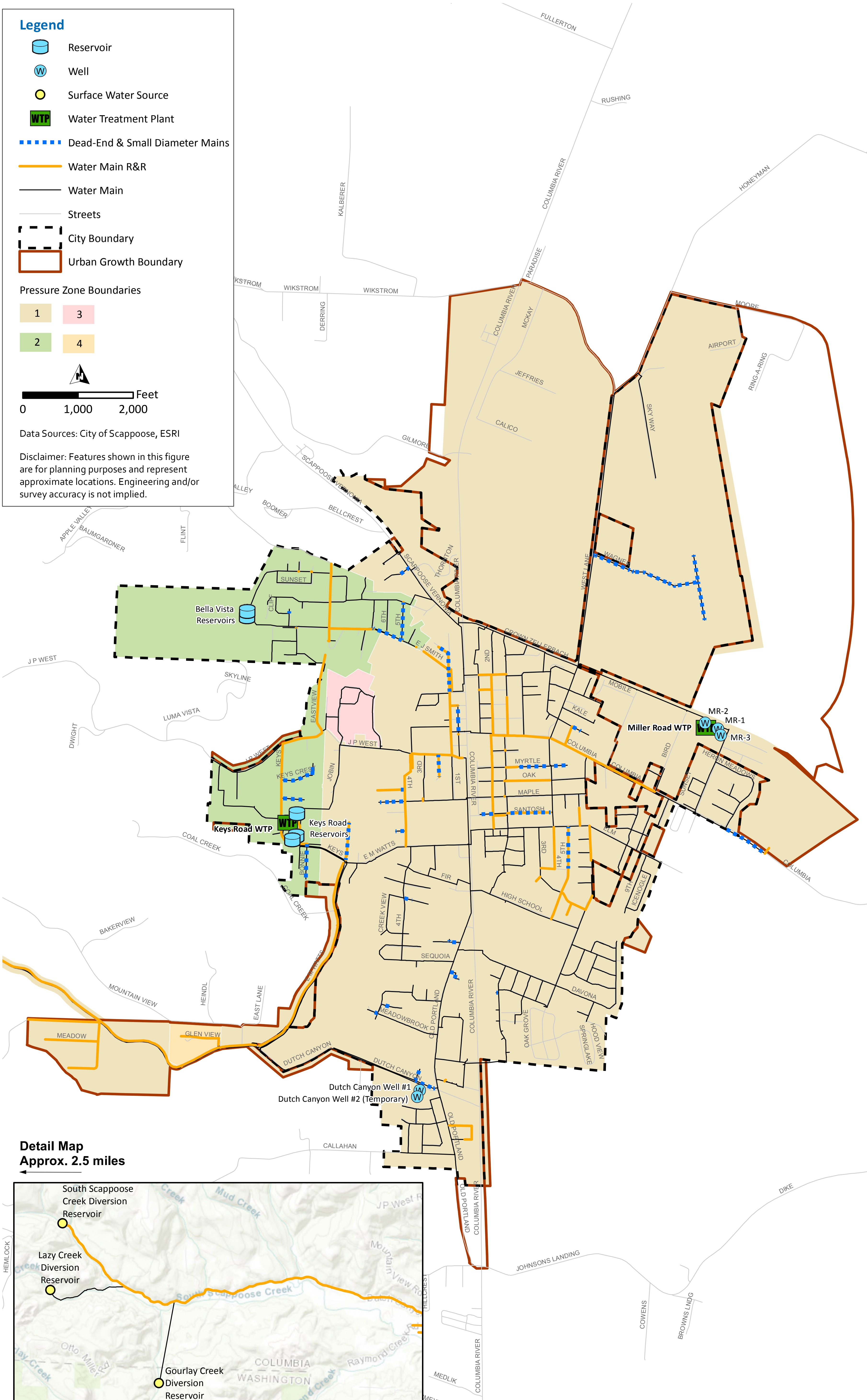


Figure 8.2 Distribution Repair and Replacement CIP Projects

8.3.3.4 Dutch Canyon Road to Em Watts Road (D-04)

Construct a 12-inch main looping SW Dutch Canyon Rd. from approximately the 33,000 block and SW EM Watts Rd. Costs assume open-trench construction and include pavement cutting, excavation, hauling, shoring, pipe materials and installation, backfill material and installation, and pavement replacement. It is assumed the water main can be affixed to the existing bridge at a similar unit cost as open-trench construction. The total projected cost for this project is $540,000, to be completed in the long-term planning horizon.

8.3.3.5 Moore Road Airport Annex (D-05)

Construct an 18-inch main along Moore Rd. and N. Honeyman Rd. to connect the existing 18-inch main along N. Honeyman Rd. at Skyway Dr. Costs assume open-trench construction and include pavement cutting, excavation, hauling, shoring, pipe materials and installation, backfill material and installation, and pavement replacement. The total projected cost for this project is $1,630,000, to be completed in the long-term planning horizon.

8.3.3.6 Airport Annex North of Bird Rd (D-06)

Construct a 12-inch main north of Bird Rd. to connect Skyway Dr. to provide looping for the E. Airport development. Costs assume open-trench construction and include pavement cutting, excavation, hauling, shoring, pipe materials and installation, backfill material and installation, and pavement replacement. The total projected cost for this project is $610,000, to be completed in the long-term planning horizon.

8.3.3.7 Water Main Repair and Replacement (D-07)

Annual program to repair and replace distribution system piping reaching the end of its useful life, which is documented in Chapter 7. Individual projects will be identified by City staff annually. To more cost-effectively address pipe replacement, consider geographically concentrated projects that address multiple concerns and incorporate other utilities, such as sewer main projects and roadway resurfacing.

8.3.3.8 Dead-End and Small Diameter Mains (D-08)

Annual program to upsize or loop dead-end or small diameter mains that were identified as deficient in Chapter 6. Individual projects will be identified by City staff annually. To more cost-effectively address pipe replacement, consider geographically concentrated projects that address multiple concerns and incorporate other utilities, such as sewer main projects and roadway resurfacing.

8.3.4 Pump Station Projects

Capital improvements to the City's High Zone booster pump station (BPS) are needed to provide additional supply as the zone develops further.

8.3.4.1 High Zone BPS (PS-01)

Construct a new BPS on the Keys Road WTP site consisting of three 25 horsepower (HP) Pumps with a firm capacity of approximately 330 gallons per minute (gpm). The new BPS will provide the needed firm capacity, while avoiding interruption of service during construction and providing a seismically resilient structure. Costs include site work, pumps, a structure, all mechanical and electrical equipment. No backup generation was included, as the existing back-up generator at the Keys Road WTP was assumed to be sufficient. No land costs were included. The total projected cost for this project is $480,000.

8.3.5 Storage Projects

8.3.5.1 2.0 MG Keys Road Reservoir (ST-01)

To meet future storage needs, the City will develop a new 2 million gallon (MG) reservoir on the Keys Road WTP site, creating a third Low Zone reservoir at the site. The reservoir costs include site work, a structure, mechanical and electrical equipment, and yard piping. No land or piping to connect to the distribution system was included in the costs. The total projected cost for this project is $4,356,000.

8.3.5.2 Reservoir Seismic Retrofit (ST-02)

This project is a placeholder for seismic mitigation improvements to the Bella Vista and Keys Road reservoirs to be identified in a forthcoming seismic study. Individual projects equal to $900,000 were assumed to be generated by the project.

8.3.6 Miscellaneous Projects

8.3.6.1 City's Capital Outlay Projects (Misc-01)

This project covers capital outlay projects already identified by the City. The City selects projects annually based on need. Costs for 2018 and 2019 include City budget estimates for capital outlay projects (City budget category 300). For 2020 through the end of the long-term planning period, an annual cost of $250,000 was included for this project.

8.4 CIP Summary

The City's complete CIP is presented in Table 8.2. Projects are listed in individual years for the short-term period, 2018 through 2028. Projects in the long-term, after 2028, are not scheduled to individual years; therefore, they are shown as a single combined long-term total. Table 8.1 provides a breakdown of the CIP by planning period and project type. Figure 8.3 provides a summary of CIP projects by project type. The short-term CIP cost is $18,786,000, or $1,708,000 per year. The long-term CIP cost is $36,340,000 or $3,634,000 per year.

Table 8.1 CIP Summary by Planning Period

Project Type	Short-term (2018-2028)	Long-term (2029-2038)	Total CIP Cost Estimate
Capacity	$11,592,000	$16,305,000	$27,897,000
Repair & Replacement	$2,224,000	$18,430,000	$20,654,000
Upgrade	$4,970,000	$1,605,000	$6,575,000
Total Cost	**$18,786,000**	**$36,340,000**	**$55,126,000**
Annual Cost	**$1,708,000**	**$3,634,000**	**$2,625,000**

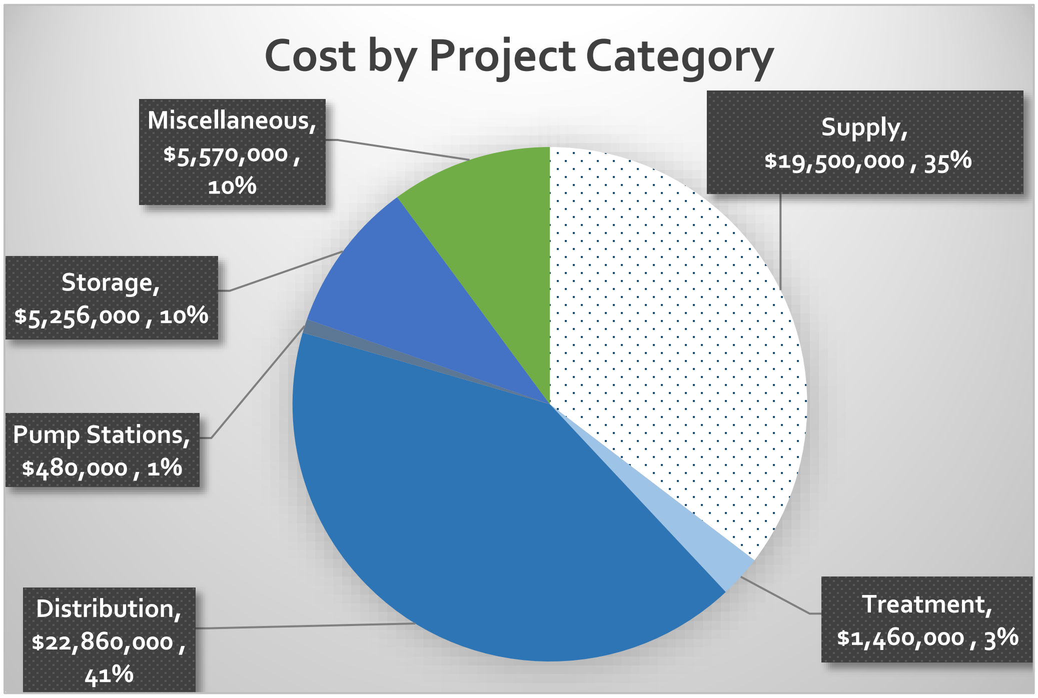


Figure 8.3 Summary of CIP Projects

Table 8.2 Summary of CIP Projects

Project	Cost Type: Current Dollars	Total CIP Cost Estimate	CIP Phasing (Current Dollars)													Developer Share (%)	Project Type		
			2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	Short-term (2018-2028)	Long-term (2029-2038)		Capacity	Repair & Replacement	Upgrade
Supply		**$19,500,000**	**$480,000**	**$0**	**$450,000**	**$390,000**	**$1,580,000**	**$420,000**	**$1,680,000**	**$420,000**	**$1,680,000**	**$0**	**$0**	**$7,100,000**	**$12,400,000**				
S-01	Dutch Canyon Well #2	$480,000	$480,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$480,000	$0	0%	100%	0%	0%
S-02	Dutch Canyon Well #3	$2,100,000	$0	$0	$0	$0	$0	$420,000	$1,680,000	$0	$0	$0	$0	$2,100,000	$0	0%	0%	0%	100%
S-03	Miller Road Well #4	$200,000	$0	$0	$200,000	$0	$0	$0	$0	$0	$0	$0	$0	$200,000	$0	100%	100%	0%	0%
S-04	Miller Road Well #5	$1,970,000	$0	$0	$0	$390,000	$1,580,000	$0	$0	$0	$0	$0	$0	$1,970,000	$0	0%	100%	0%	0%
S-05	Miller Road Well #6	$2,100,000	$0	$0	$0	$0	$0	$0	$0	$420,000	$1,680,000	$0	$0	$2,100,000	$0	0%	100%	0%	0%
S-06	Long-Term Supply	$12,650,000	$0	$0	$250,000	$0	$0	$0	$0	$0	$0	$0	$0	$250,000	$12,400,000	0%	100%	0%	0%
Treatment		**$1,460,000**	**$0**	**$410,000**	**$100,000**	**$0**	**$0**	**$600,000**	**$350,000**	**$0**	**$0**	**$0**	**$0**	**$1,460,000**	**$0**				
T-01	Miller Road R&R	$650,000	$0	$150,000	$0	$0	$0	$500,000	$0	$0	$0	$0	$0	$650,000	$0	0%	0%	100%	0%
T-02	Keys Road R&R	$340,000	$0	$240,000	$100,000	$0	$0	$0	$0	$0	$0	$0	$0	$340,000	$0	0%	0%	100%	0%
T-03	Supply and Treatment Plant LOS Goals	$20,000	$0	$20,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$20,000	$0	0%	0%	0%	100%
T-04	Seismic and Life-Safety Audit	$160,000	$0	$0	$0	$0	$0	$60,000	$100,000	$0	$0	$0	$0	$160,000	$0	0%	0%	100%	0%
T-05	Treatment Capacity and Operations Optimization Study	$290,000	$0	$0	$0	$0	$0	$40,000	$250,000	$0	$0	$0	$0	$290,000	$0	0%	0%	0%	100%
Distribution		**$22,860,000**	**$0**	**$60,000**	**$60,000**	**$60,000**	**$60,000**	**$110,000**	**$310,000**	**$810,000**	**$60,000**	**$60,000**	**$60,000**	**$1,650,000**	**$21,210,000**				
D-01	NW Eastview Drive Replacement	$790,000	$0	$0	$0	$0	$0	$0	$200,000	$590,000	$0	$0	$0	$790,000	$0	0%	40%	60%	0%
D-02	SW 5th Street Connection	$210,000	$0	$0	$0	$0	$0	$0	$50,000	$160,000	$0	$0	$0	$210,000	$0	0%	100%	0%	0%
D-03	Sky Way Drive Connection Airport Annex	$50,000	$0	$0	$0	$0	$0	$50,000	$0	$0	$0	$0	$0	$50,000	$0	100%	100%	0%	0%
D-04	Dutch Canyon Rd to Em Watts Rd	$540,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$540,000	0%	100%	0%	0%
D-05	Moore Rd Airport Annex	$1,630,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$1,630,000	0%	100%	0%	0%

Table 8.2 Summary of CIP Projects (continued)

Project	Cost Type: Current Dollars	Total CIP Cost Estimate	CIP Phasing (Current Dollars)													Developer Share (%)	Project Type		
			2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	Short-term (2018-2028)	Long-term (2029-2038)		Capacity	Repair & Replacement	Upgrade
Distribution (continued)		**$22,860,000**	**$0**	**$60,000**	**$60,000**	**$60,000**	**$60,000**	**$110,000**	**$310,000**	**$810,000**	**$60,000**	**$60,000**	**$60,000**	**$1,650,000**	**$21,210,000**				
D-06	Airport Annex North of Bird Rd	$610,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$610,000	0%	100%	0%	0%
D-07	Water Main Repair and Replacement	$15,500,000	$0	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$500,000	$15,000,000	0%	0%	100%	0%
D-08	Dead-End and Small Diameter Mains	$3,530,000	$0	$10,000	$10,000	$10,000	$10,000	$10,000	$10,000	$10,000	$10,000	$10,000	$10,000	$100,000	$3,430,000	0%	0%	100%	0%
Pump Stations		**$480,000**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$480,000**				
PS-01	High Zone BPS	$480,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$480,000	0%	0%	0%	100%
Storage		**$5,256,000**	**$0**	**$0**	**$0**	**$900,000**	**$0**	**$0**	**$0**	**$0**	**$0**	**$1,089,000**	**$3,267,000**	**$5,256,000**	**$0**				
ST-01	2.0 MG Keys Road Reservoir	$4,356,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$1,089,000	$3,267,000	$4,356,000	$0	0%	100%	0%	0%
ST-02	Reservoir Seismic Retrofit	$900,000	$0	$0	$0	$900,000	$0	$0	$0	$0	$0	$0	$0	$900,000	$0	0%	0%	0%	100%
Miscellaneous		**$5,570,000**	**$585,000**	**$485,000**	**$250,000**	**$250,000**	**$250,000**	**$250,000**	**$250,000**	**$250,000**	**$250,000**	**$250,000**	**$250,000**	**$3,320,000**	**$2,250,000**				
Misc-01	City's Capital Outlay Projects	$5,570,000	$585,000	$485,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$250,000	$3,320,000	$2,250,000		50%	0%	50%
CIP Total (Current Dollars)		**$55,126,000**	**$1,065,000**	**$955,000**	**$860,000**	**$1,600,000**	**$1,890,000**	**$1,380,000**	**$2,590,000**	**$1,480,000**	**$1,990,000**	**$1,399,000**	**$3,577,000**	**$18,786,000**	**$36,340,000**		**$27,897,000**	**$20,654,000**	**$6,575,000**
Annual Cost (Current Dollars)		**$2,625,000**	**$1,065,000**	**$955,000**	**$860,000**	**$1,600,000**	**$1,890,000**	**$1,380,000**	**$2,590,000**	**$1,480,000**	**$1,990,000**	**$1,399,000**	**$3,577,000**	**$1,708,000**	**$3,634,000**		**$1,328,000**	**$984,000**	**$313,000**

Chapter 9

FINANCIAL PLAN

9.1 Introduction

This chapter summarizes the financial status of the City of Scappoose (City) and provides a cursory evaluation of the City's ability to finance the necessary capital improvements identified in the Capital Improvement Plan (CIP) as outlined in Chapter 8. Financial status of the City's water utility, funding required to finance the scheduled improvements, potential funding sources, and the impact of water system improvements on water rates are presented.

9.2 Historical Financial Performance

9.2.1 Rates

The City has the following rate categories for their water system customers, based on water meter size:

- 3/4-inch or 1-inch meter, Inside City.
- 1.5- to 2-inch meter (no maximum fire flow).
- 1.5- to 2-inch meter (with maximum fire flow).
- 3-inch meter.
- 4-inch or above meter.
- Outside City.
- 3/4-inch Dutch Canyon Service.

9.2.1.1 Monthly Fixed Fee

The following table shows the City's current fixed base rates, effective July 16, 2018. Per Resolution 17-16, monthly rates for water shall be adjusted on July 16 of each year, and be adjusted by the annual change in Construction Cost Index (CCI) according to the Engineering News-Record (ENR) 20 City Average published in the ENR in December of each year, unless otherwise adjusted by the City.

Table 9.1 2018 Monthly Base Fee

Rate Category	Monthly Charge
3/4- or 1-inch Inside City	$32.98
1.5 to 2.0-inch meter (No fire flow)	$144.14
1.5 to 2.0-inch meter (with fire flow)	$243.02
3.0-inch meter	$675.49
4.0-inch meter and above	$947.29
3/4-inch Dutch Canyon	$37.27
Outside City	$74.17

Note:
(1) Source: City 2018 Rate Schedule, adopted July 16, 2018.

9.2.1.2 Commodity Rate

In addition to the monthly charge, customers also pay a usage charge for the water consumed. The following table shows the monthly commodity rate the City charges its water customers.

Table 9.2 2018 Commodity Rates

Consumption, (gallons)	$/100 gallons
1 to 5,000	$0.38
5,001 to 7,500	$0.38
7,501 to 10,000	$0.42
10,001 +	$0.43

Note:
(1) Source: City 2018 Rate Schedule, adopted July 16, 2018.

9.2.2 Historical Financial Operations

The City's operating revenues for the years 2014 to 2018 are summarized in Table 9.3. This information was provided through the City's financial statements and includes the City's system development charge (SDC) revenue but not the beginning fund balance for that year. Table 9.4 shows the City's historical expenses for the same time period, and includes infrastructure upgrades. Figure 9.1 shows a graphical representation of the historical revenues and expenses; the City has been able to cover operational expenses through water sales.

Table 9.3 Historical Operating Revenue

Operating Revenue	2014	2015	2016	2017	2018 Estimated
Water Sales	$1,332,513	$1,562,595	$1,872,232	$1,949,200	$2,073,000
Other Revenue(2)	206,130	967,593	152,696	830,027	456,896
Total	**$1,538,643**	**$2,530,188**	**$2,024,928**	**$2,779,226**	**$2,529,896**

Notes:
(1) Source: City's financial statements.
(2) Other revenue includes interest, SDCs, intergovernmental, and miscellaneous revenue.

Table 9.4 Historical Operating Expenses

Operating Expenses	2014	2015	2016	2017	2018
Operating Expenses(2)	$1,061,823	$1,318,683	$1,260,775	$1,773,308	$1,359,563
Debt Service Payments	358,737	361,787	359,588	362,332	359,783
Total	**$1,420,560**	**$1,680,470**	**$1,620,363**	**$2,135,640**	**$1,719,346**

Notes:
(1) Source: City's financial statements.
(2) Operating expenses include materials, services, infrastructure upgrades, and transfers out. Does not include depreciation.

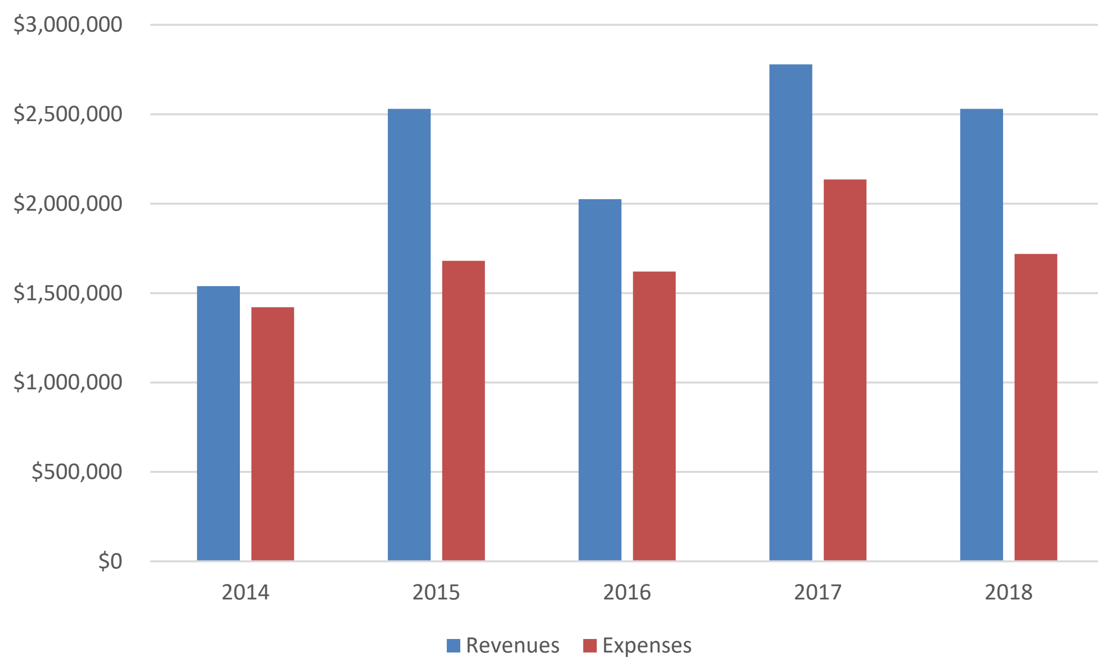


Figure 9.1 Historical Revenues vs Expenses

9.2.3 Outstanding Debt

The City has the following outstanding loans:

- *Loan S03003B-40-400* – 20-year principal and interest payment of approximately $81,439 per year, ending in 2030.
- *Miller Road Water Plant G03003 50-500* – 25-year principal and interest payment averaging approximately $121,000 per year, ending in 2028.
- *Water Storage S03003 50-500* – 30-year principal and interest payment of approximately $157,800 per year, ending in 2036.
- Miscellaneous Equipment and Vehicle Leases.

The City's Debt Service Coverage Ratio (DSCR) is calculated by dividing the City's net income (revenues less expenses) by the annual debt service payment. The City currently has a DSCR of 2.10.

9.2.4 System Development Charges

SDCs are one-time charges paid by new development to reimburse existing utility customers for costs previously paid to construct current system capacity or to help finance planned future growth-related capacity improvements. The charges help ensure that all customers connecting to the system bear an equitable share of costs that have been or will be invested to provide capacity needed to serve them and any further growth-related expansion. SDC revenues are deposited in the utility's capital fund and are used to help support current and future capital expenditures.

Water SDC rates are traditionally recalculated following the update of the Water System Master Plan Update (Plan) to reflect historical capital investments through that date and capital expenditures outlined in the Plan that are needed to support future growth.

Per Resolution 17-17 that was adopted May 15, 2017, "The adjustment factor shall be based on the annual change in construction costs according to the ENR 20 City Average CCI published in the ENR in December of each year." For this analysis, the SDC inflation rate of 3.0 percent for projected years was calculated by taking the average of the December percent change for the past 10 years (2008-2017).

Using the growth rate assumption of 4.0 percent and the SDC inflation rate of 3.0 percent, this analysis projects the City's SDC revenue will increase considerably more than what they are currently receiving. It is recommended to closely monitor the SDC revenue in the coming years, as any revenue adjustments could affect the City's CIP schedule.

9.3 Financial Forecast

The financial forecast provides the City with a snapshot of their current financial status. As there are numerous assumptions presented in this analysis, the projected results can vary from the actual data depending on factors like actual customer use, demand projection, and growth. Therefore, this high-level projection should be later compared with actuals and adjusted accordingly. For planning purposes, the City was shown a forecast of the "status quo," or if rate increases were adjusted per the resolutions and no debt was issued, and then shown the following two scenarios that would help the City choose the best path forward to fund the projected CIP shown in Chapter 8:

- **Scenario 1**: Annual rate increases to cover CIP; no new debt issued.
- **Scenario 2:** Annual 3.0 percent rate increases; new debt issued for certain projects; $2 million in reserves.

9.3.1 Projected Cash Flow (Status Quo)

Table 9.5 shows the ten-year projected operating cash flow for the water utility from a "status quo" perspective. This assumes that water sales and SDC rate increases would continue, per the City's resolutions, at an assumed CCI inflation rate of 3.0 percent per year. Other assumptions included an annual water growth rate of 4.0 percent, the projected CIP is not adjusted, no new debt is issued, and there are no loans or grants applied. The table subtracts the total operation and maintenance expenses, existing debt, and the projected CIP from the total operating revenues and fund balance. The result shows that by 2022, the ending fund balance falls below the $2 million reserve requirement and the City's cash flow goes negative by 2024 and would not have the funds to cover basic operating expenses or the projected CIP.

9.3.1.1 Rate Increases

The projected water sales shown in Table 9.5 include an "across the board" rate revenue increase, which assumes the revenue collected from the metered water sales the previous year will increase by a certain percentage and is not calculated by each customer category. For Scenario 1, a 5.5 percent rate increase is assumed from 2019 to 2028. Scenario 2 shows a 3.0 percent increase from 2019 to 2028 along with certain debt issued in order to keep the City's minimum requirement of operating reserves at $2 million while funding the proposed CIP.

Using forecast factors from the City and assumptions from the demand projections described earlier in this Plan, operating revenues and expenditures were projected to 2028 and are shown in Table 9.5. These projections are for Scenario 1 and do not include the projected CIP.

9.3.1.2 Projected Capital Improvement Projects

Table 9.6 presents the water CIP based on the short-term (2019 to 2028) projects listed in Chapter 8. For the purpose of this analysis, the projects were designated by project categories and further broken out by the following types of projects:

- **Capacity Projects:** These projects are completed to meet future system growth and typically funding by SDCs.
- **Repair and Replacement Projects:** These projected are for replacing or maintaining the utility's existing infrastructure and are funded by reserves and rate revenue.
- **Upgrade Projects:** Projects that increase the level-of-service of existing infrastructure and are typically funded by rate revenue.

The projects were then allocated to certain years within the 2019-2028 period. These designations are assumptions and can be shifted as the utility determines a more detailed schedule. The project costs are in future dollars. The financial model shows that for Scenario 1, the utility will have to use reserves in order to pay for the projected capital projects. By the end of the 10-year planning period, the reserves will have approximately $3.6 million remaining in the fund balance.

Table 9.5 Projected Cash Flow (short-term)

	2019 Budget	2020	2021	2022	2023	2024	2025	2026	2027	2028
Beginning Fund Balance	$2,154,822	$2,130,360	$2,210,738	$2,081,738	$1,336,318	$1,351,299	($364,358)	($111,875)	($292,434)	$486,201
Total Operating Revenue	3,012,060	3,082,010	3,302,270	3,536,090	3,780,690	4,052,370	4,329,790	4,639,420	4,968,940	5,322,600
(-) Total Operating Expenses	1,684,365	1,727,400	1,757,028	1,797,558	1,796,320	1,848,200	1,898,280	1,949,990	2,004,590	2,060,600
(-) Total Debt	$362,158	$359,232	$361,242	$362,952	$359,389	$360,827	$362,027	$362,989	$358,714	$359,440
(-) Total CIP	990,000	915,000	1,313,000	2,121,000	1,610,000	3,559,000	1,817,000	2,507,000	1,827,000	4,806,000
Ending Fund Balance	**$2,130,360**	**$2,210,738**	**$2,081,738**	**$1,336,318**	**$1,351,299**	**($364,358)**	**($111,875)**	**($292,434)**	**$486,201**	**($1,417,238)**

Notes:
(1) Source: City financial data.
(2) Assumes "across the board" rate increases for water sales and SDCs per the City's Resolutions 17-16 and 17-17.

Table 9.6 Projected Capital Improvement Projects (short-term)

	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	Total
Capacity Projects	$772,500	$250,000	$612,500	$566,500	$1,910,500	$205,000	$301,500	$1,161,500	$2,278,500	$1,584,000	$9,642,500
Repair and Replacement Projects	$0	$470,000	$170,000	$60,000	$70,000	$720,000	$808,000	$502,000	$70,000	$80,000	$2,950,000
Upgrade Projects	$292,500	$270,000	$132,500	$686,500	$140,500	$685,000	$2,449,500	$153,500	$158,500	$163,000	$5,131,500
Total	**$1,065,000**	**$990,000**	**$915,000**	**$1,313,000**	**$2,121,000**	**$1,610,000**	**$3,559,000**	**$1,817,000**	**$2,507,000**	**$1,827,000**	**$17,724,000**

Note:
(1) See Chapter 8 for detailed project information.

9.4 Financial Forecast Scenarios

The two scenarios presented to the City are shown in the following subsections. Each year on the bar graph shows the amount of operational expenses, the projected CIP, and any debt payments. The amount of projected revenue the City will have each year is represented in the line graph, and is related to the rate increase percentages shown in the x-axis. Finally, the Ending Fund Balance is shown in another line graph in order to understand how much reserves are projected to remain at the end of the year.

9.4.1 Scenario 1: Rate increases, No Debt Issued

Figure 9.2 shows Scenario 1's projected financial forecast. A 5.5 percent annual rate increase was included in order to cover the CIP. The ending fund balance remains above $2 million except in 2024, when there is a larger projection of projects to be completed.

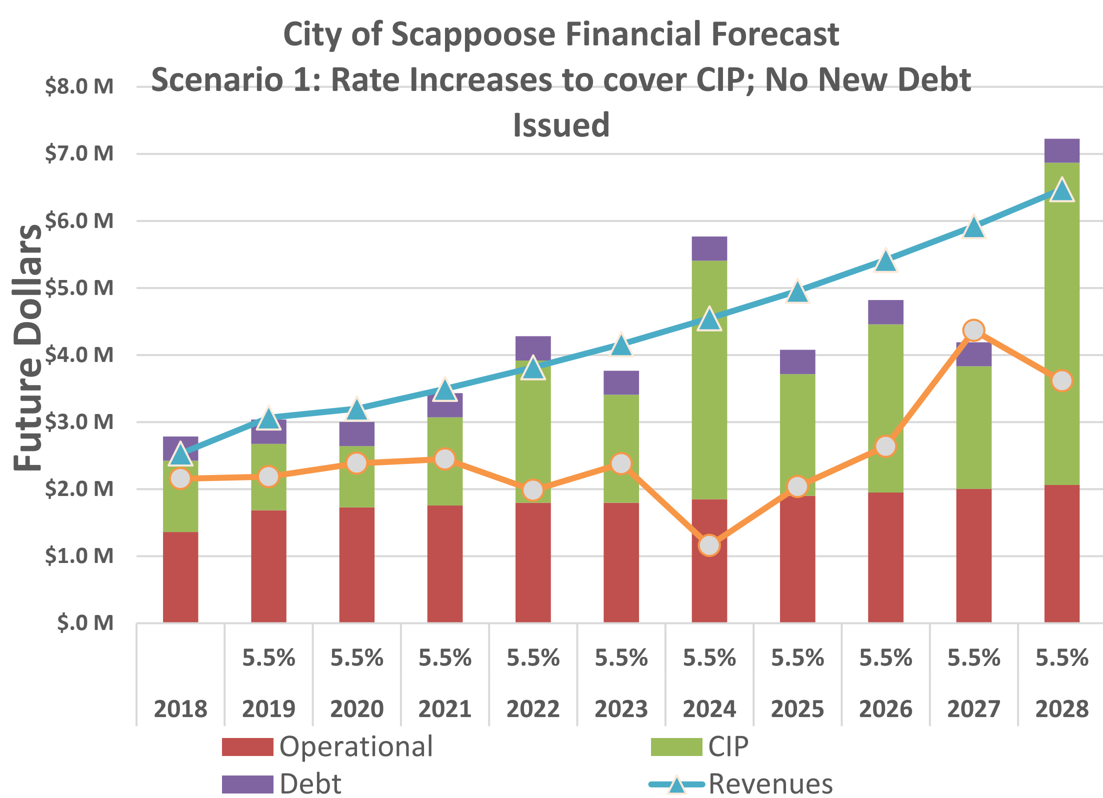


Figure 9.2 Financial Forecast: Scenario 1

9.4.2 Scenario 2: Rate increases; Some Debt Issued; $2 million Reserves

Figure 9.3 shows Scenario 2's financial forecast. Similar to Scenario 1, annual rate increases were necessary in order to cover future projects. In this scenario, debt was also issued to fund three major projects in order to keep rate increases to 3.0 percent per year. The three projects are:

- S-02: Miller Road Well #5 (partially funded by the Oregon Manufacturing Innovation Center [OMIC]).
- S-04: Dutch Canyon Well #4 or a comparable supply project.

- ST-01: New 2 million gallon Keys Road Reservoir (partially funded by OMIC).

Another criteria from the City was to keep the reserves above $2 million, which is shown as the dashed line.

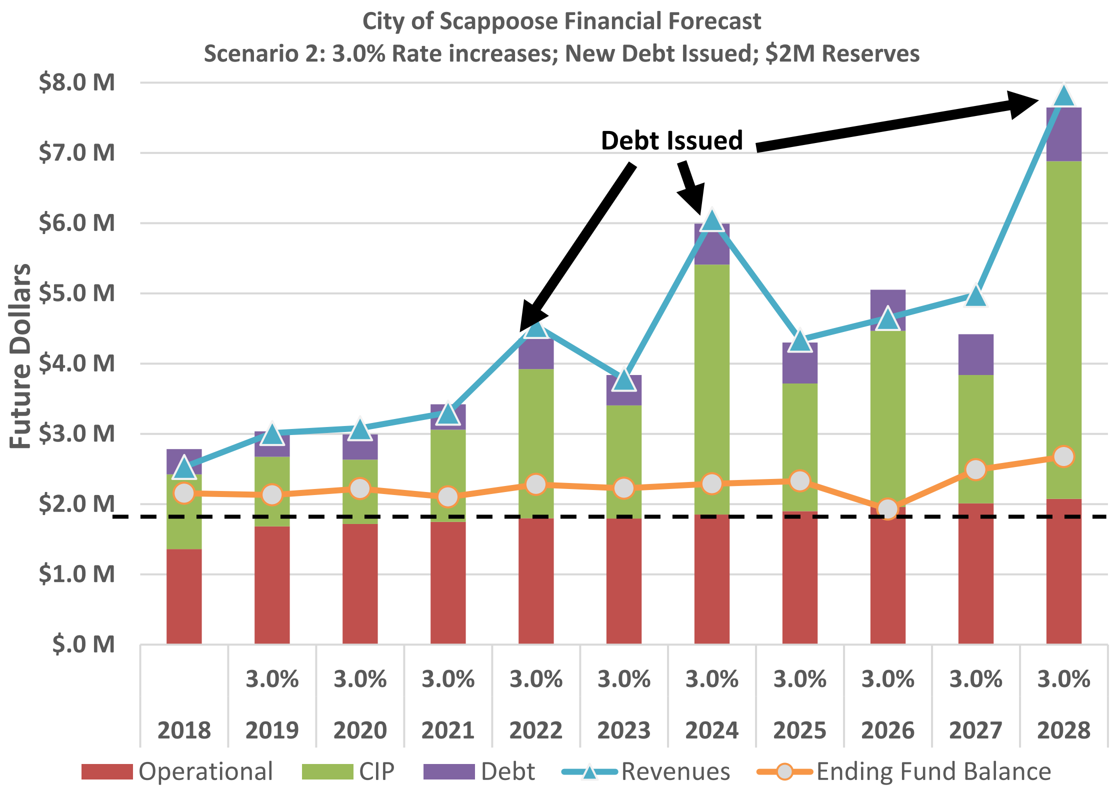


Figure 9.3 Financial Forecast: Scenario 2

9.5 Available Funding Assistance and Financing Resources

The ten-year planning period shows the City's ending fund balance will be adequate to fund the anticipated short-term projects shown in the CIP, provided there are moderate rate increases in Scenario 1. However, should the City issue more debt to keep the rate increases lower, as is shown in Scenario 2, it is important for the City to understand what funding assistance and financing resources are available to help fund the CIP should reserves fall to a level that would not fund the CIP. The following is a summary of the City's resources.

9.5.1 Oregon Department of Environmental Quality

Per the Oregon Department of Environmental Quality (DEQ) website, low-cost loans for planning, design, and construction projects can be found through this department and offer the following:

- Low-Cost Loans and Bond Purchases.
- Lower Than Market Interest Rates.
- Fixed Interest Rates.
- Terms Up To 30 Years.

- Up To 100 Percent Of Eligible Costs Covered.
- No Match Required.
- Repayment Begins After Project Is Constructed.
- No Pre-Payment Penalty.
- Additional Financial Incentives, Including Principle Forgiveness.

9.5.2 Grants and Low Cost Loans

The Business Oregon – Infrastructure Finance Authority (IFA) and the Oregon Water Resources Department (WRD) offer grants and loans for water projects. The following types of loans and grants are available through these departments:

- Safe Drinking Water Revolving Loan Fund (SDWRLF).
- Drinking Water Source Protection Fund (DWSP).
- Community Development Block Grant (CDBG).
- Water Project Grants and Loans through WRD.

9.5.3 Bond Financing

Bond financing is obtained by issuing general obligation or revenue bonds. Revenue bonds do not require voter approval and may be repaid with revenues from rates, miscellaneous fees or connection charges.

9.6 Summary

Upon analysis of the financial status of the water utility, the City has adequate revenues from water rates and system development charges to meet the expected operating costs of the water system through 2028, provided additional rate increases above the 3.0 percent inflation rate are implemented and the projected growth assumptions are met. Capital projects are projected to average approximately $2.5 million annually in the next ten years, which will require the utility to use reserves to fund these projects. If the City chooses not to issue any debt, water rates are anticipated to increase 5.5 percent per year for the next ten years. The ending fund balance would remain above the $2 million reserve requirement except during 2024. The City also has the option to issue debt, as shown in Scenario 2, that would help lower the annual rate increases over the next ten years to 3.0 percent, and issuing debt in three years to cover the larger anticipated projects as noted in Chapter 8. The City can also apply for qualifying grants and loans for the larger projects. It is recommended to conduct a rate and cost of service study to determine the appropriate rate increases for each customer class, which would help cover the costs of the projected capital projects. It is also recommended to complete an SDC study to confirm the projected charges are adequate for new development.

Chapter 10

SEISMIC RISK ASSESSMENT AND MITIGATION PLAN

10.1 Introduction

As part of the Water System Master Plan Update (Plan), the Oregon Health Authority (OHA) Drinking Water Services requires water systems with over 300 connections to prepare a seismic risk assessment and mitigation plan, using the 2013 Oregon Resilience Plan as a road map for earthquake preparedness. This seismic assessment and mitigation plan has two goals:

1. Identify critical infrastructure needed to supply water during an emergency.
2. Identify improvements to supply, pumping, storage, and distribution so customers are provided with water following a Cascade subduction zone earthquake.

This chapter identifies seismic hazards within the City of Scappoose (City) and defines the water system's seismic system, including critical facilities and components that are needed in order to continue to supply water to the community's essential needs after a Cascadia Subduction Zone (CSZ) earthquake. This chapter includes a summary of seismic vulnerability assessments performed to understand potential performance of the City's water treatment system to the M9.0 CSZ earthquake. This chapter also identifies the likelihood and consequences of seismic failures for each facility and makes recommendations to improve seismic resilience, which will be integrated into a 50-year Mitigation Plan. An Executive Summary can be found at the end of this chapter, which highlights the findings of this assessment.

The assessments were performed to identify deficiencies that, if addressed, would increase system resilience. Recommended studies will better enable the City to evaluate the likelihood and consequences of seismic failure for each critical facility, and provide a 50-year mitigation plan that identifies what components of the system will need upgrades to meet Oregon Resilience Plan level of service goals.

The seismic vulnerability assessments were conducted by geotechnical and structural engineers that reviewed available information for the components assessed; inspected accessible portions of structures, facilities, and the conveyance system; performed structural analysis of buildings, tanks, and structures; and used the American Society of Civil Engineers (ASCE) 41-13 Seismic Evaluation and Retrofit of Existing Buildings (ASCE 41) Tier 1 screening procedures and the ASCE Technical Committee on Lifelines Earthquake Engineering Monograph 22 (TCLEE-22) Seismic Screening Checklists for Water and Wastewater Facilities. The team included professionals from Carollo Engineers, Inc. (Carollo) who assessed buildings and structures, and non-structural components, In addition, McMillan Jacobs Associates (MJA) performed a geotechnical hazard evaluation for the assessments.

This Chapter provides notes of the water treatment plant (WTP) buildings and a summary of the nonstructural systems. Additional detailed notes of the assessments performed by the team members are included in appendix:

- Appendix A – Seismic Hazards Evaluation (MJA).

The structures and buildings at the WTPs and pump stations that were assessed are summarized in Table 10.1.

Table 10.1 Structures and Buildings Assessed

Structure Name	Original Construction	Modification(s)
Water Treatment Plants		
Keys Road Treatment Building	1979	2000(1)
Miller Road Treatment Building	2004	---
Reservoirs		
Keys Road 1, 2 MG Reservoir(3)	2004	--
Keys Road 2, 1 MG Reservoir	1967	--
Keys Road 3, 0.2 MG Reservoir	1947	--
Bella Vista 1, 0.30 MG	1967	--
Bella Vista 2, 0.37 MG	2003	--
Pump Stations		
Keys Road	1979	--
Glen View	2010	--
Wells		
Miller Road 1 Emergency Well	2002	--
Miller Road 2 Production Well	2004	--
Miller Road 3	2004	--
Dutch Canyon	1979	--

Notes:
(1) Addition of Filter 3.
(2) MG: million gallons.

10.2 Seismic Hazard Assessment

Seismic hazards include strong ground shaking, liquefaction settlement, lateral spreading, and seismically induced landslides. These hazards can damage facilities such as pipelines or above-ground structures through either ground deformation or intense shaking.

To identify seismic hazards within the City's system for a magnitude 9.0 CSZ scenario, MJA performed a seismic hazards assessment using data sets published by the Oregon Department of Geology and Mineral Industries (DOGAMI) and historic boring records and site reconnaissance.

The following sections summarize this assessment. For further details on the development on this data, refer to TM 1 – Seismic Hazard Evaluation (Appendix A).

10.2.1 Seismic Hazard Findings

The following sections detail the results of the seismic hazard evaluation. Table 1 in TM 1 - Seismic Hazard Evaluation provides a preliminary seismic hazard assessment summary for the critical facilities.

10.2.1.1 Miller Road Water Treatment Plant

There are no geotechnical seismic concerns at the Miller Road WTP site. The risk of liquefaction, lateral spreading and landslide is negligible. This assessment is based on well house geotechnical information and well logs. No further action is required at this time.

10.2.1.2 Dutch Canyon Well

There are no geotechnical seismic concerns at the Dutch Canyon Well site. The risk of liquefaction, lateral spreading and landslide is negligible. This assessment is based on the well log. No further action is required at this time.

10.2.1.3 Keys Road Water Treatment Plant and Reservoirs

Liquefaction and lateral spreading concerns were identified. Liquefaction settlement preliminary estimate is 3 to 6 inches. Lateral spreading displacement preliminary estimate is 12 to 24 inches. This assessment is based on geotechnical information used during design of the 2000 Filter addition and the 2004 2.0 MG Reservoir.

The WTP has a wooden pile foundation that was designed for gravity loads. The 2.0 MG Reservoir is built on stone columns. These deep foundations may provide structural stability in the event of liquefaction and/or lateral spreading. The other structures have mat slab foundations. Most of the foundations are close to grade. The 0.20 MG Reservoir, 1.0 MG Reservoir and Washwater Basin mat foundations are embedded between 6 and 18 feet below grade.

A site specific geotechnical evaluation is recommended to evaluate the anticipated liquefaction and lateral spreading magnitude and the seismic stability of the existing foundations.

10.2.1.4 Bella Vista Reservoirs

A potential landslide hazard was identified at the Bella Vista Reservoirs site. The risk of a seismic induced landslide is characterized as low. The risk of liquefaction or lateral spreading is negligible. This assessment is based on the geotechnical information used during design of the 0.37 MG Bella Vista No. 2 Reservoir.

10.3 City of Scappoose Seismic System

The WTP structures and pump stations identified in this Chapter are not redundant and are considered critical for the City to meet the water system performance objectives.

There are two in-service reservoirs at each site. Despite this redundancy, each reservoir was considered critical for this assessment. With further evaluation, rating one reservoir at each site as less important might be possible.

Keys Road Reservoir No. 3 is not in service. It is not considered a critical facility.

10.3.1 Seismic System Development Overview

In compliance with Oregon Administrative Rule (OAR) 333-061-0060, the seismic risk assessment must identify critical facilities needed to supply water to key community needs during a seismic event (fire suppression, health care, first aid emergency, drinking water). With input from City staff, the assessment identified the seismic system and its infrastructure, which include key supply, treatment, distribution, and storage elements required to continue supplying water to the community after a CSZ earthquake.

The City is following recommendations outlined in the 2013 Oregon Resilience Plan (ORP), which defines the seismic backbone system's function as follows: "The backbone water system would be capable of supplying key community needs, including fire suppression, health and emergency response, and community drinking water distribution points, while damage to the larger (non-backbone) system is being addressed."

10.3.2 Seismic System Criteria

The ORP presents target states of recovery after a magnitude 9.0 CSZ earthquake for critical public services, including water supply systems, for regions in the state. Figure 10.1 shows the target states of recovery for domestic water supply in the "Valley" region, where the City is located. These guidelines were used to help create the seismic system.

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		

80–90% Operational	G
50–60% Operational	Y
20–30% Operational	R
Current State/90% Operational	X

Figure 10.1 City Target States of Recovery for Domestic Water Supply

10.3.3 Seismic System Results

As seen in Figure 10.1, the ORP recommends the seismic system's main transmission facilities, pipes, pump stations, and reservoirs to be 80 to 90 percent operational within 24 hours after the M9.0 CSZ event. This means that the seismic system must be able to withstand an earthquake with little to no damage and remain pressurized. Thus, to provide realistic goals in water resilience planning, the ORP recommends a phased improvement plan that focuses efforts first on developing the seismic system so it serves its function.

The City identified a critical seismic system and is shown in Figure 10.2. The system is divided into the following sections:

1. **Section A:** From Bella Vista Reservoirs to the intersection of SW J.P. West road and SW Jobin lane.
2. **Section B:** Connects the Miller Road WTP to the Scappoose Fire Department and Scappoose Public Works via East Columbia Avenue.
3. **Section C:** Connects the Keys Road WTP and reservoirs to Section B via SW J.P. West road and SW Jobin lane.
4. **Section D:** Connects Scappoose Middle School, Scappoose High School, Otto Peterson Elementary, and Grant Watts Elementary School to the backbone system via Columbia River highway.
5. **Section E:** Connects Dutch Canyon Well No. 1 to Section D via Old Portland road.

These sections are not prioritized in any way, but simply break out the City's backbone system to help categorize and schedule future projects.

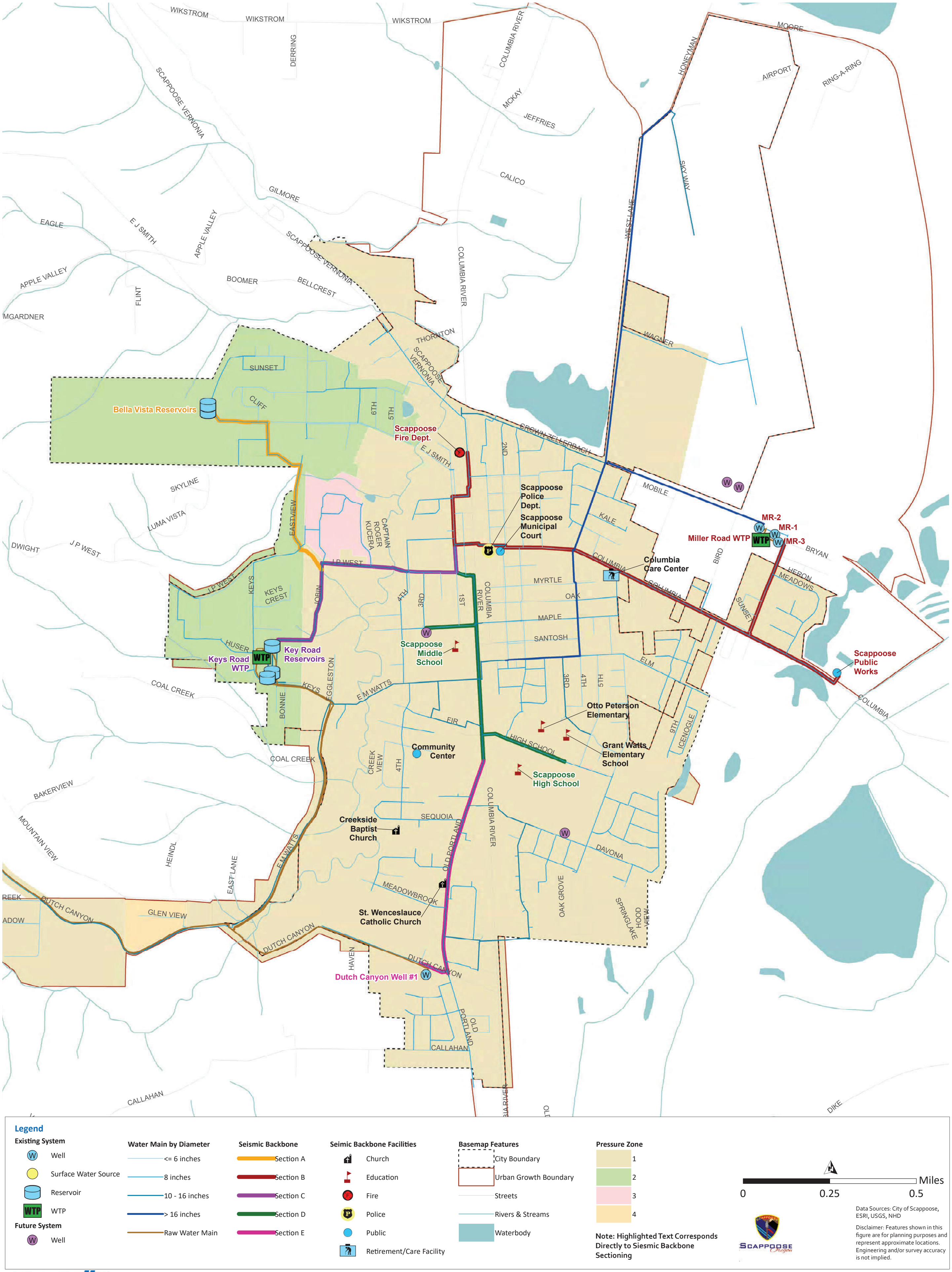
Community water distribution points and firefighting supply locations were not specifically identified for this assessment. However, we recommend locating these facilities along the seismic system and identifying additional piping to serve them.

The seismic system shown in Figure 10.2 should be periodically reevaluated as the City continues to coordinate with internal departments and regional emergency planning services, such as fire and police. Other factors that will drive potential modifications include accommodating new critical facilities, emergency shelter locations, and opportunity projects with road improvements, such as the construction of resilient bridges.

10.3.3.1 Pipeline Assessment

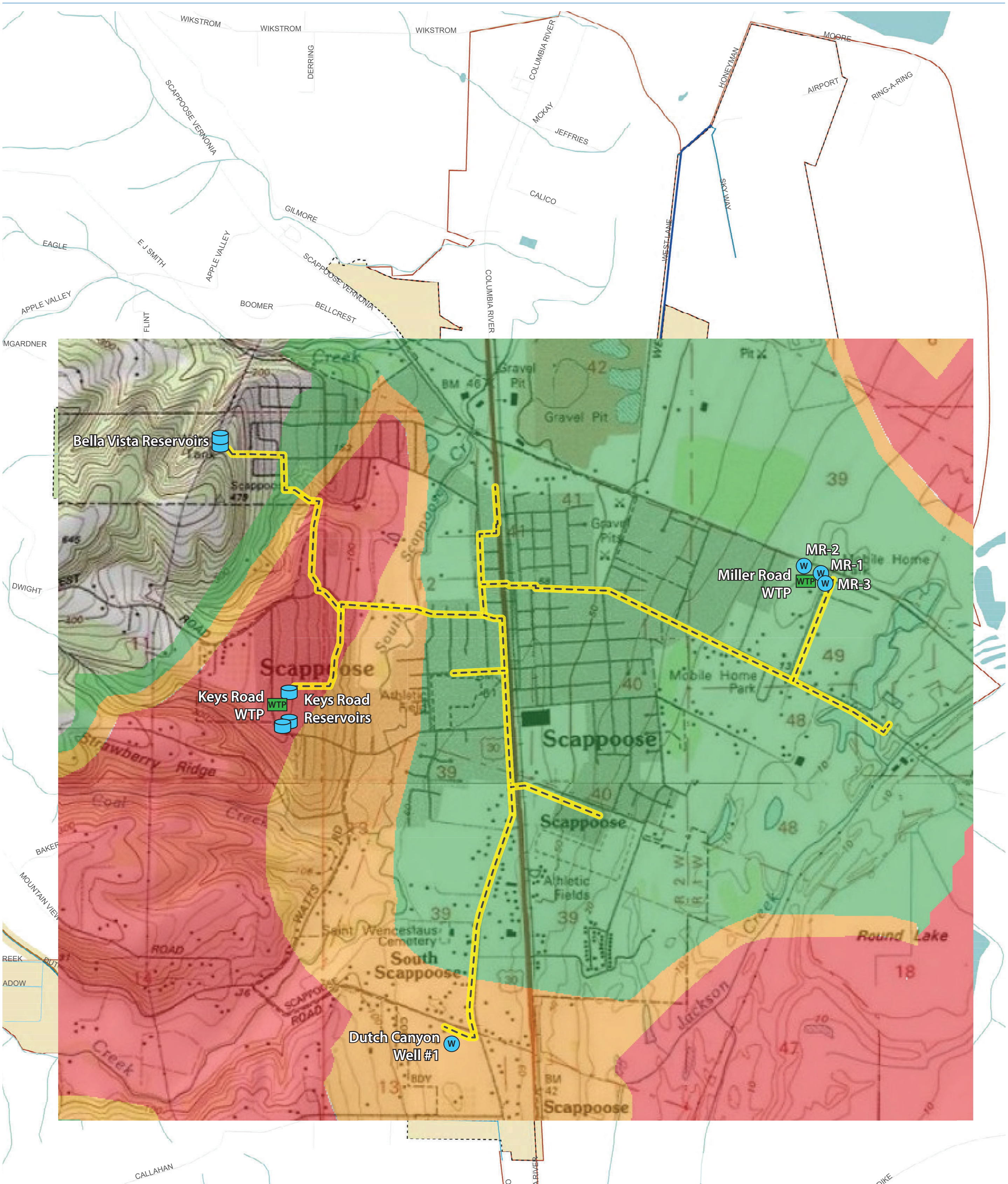
The following Figures 10.3 and 10.4 illustrate potential geologic hazards along the City's backbone system. The Oregon DOGAMI has compiled large scale mapping of geohazards for the state of Oregon. These maps at best should be considered screening tools that provide an indication of potential risks that may affect the backbone system during a seismic event.

Two potential risks previously discussed may impact the backbone system: liquefaction and landslides. Figures 10.3 and 10.4 illustrate the general risk for landslide and liquefaction by overlaying the City's backbone system on the DOGAMI HazVu mapping.



carollo

Figure 10.2 Seismic Backbone



Legend

Existing System

- Well
- Surface Water Source
- Reservoir
- WTP

Future System

- Well

LIQUEFACTION PROBABILITY

- HIGH
- MODERATE
- LOW

DATA SOURCE:
DOGAMI HAZVU -
STATEWIDE HAZARDS VIEWER

0 0.25 0.5 Miles

Data Sources: City of Scappoose, ESRI, USGS, NHD

Disclaimer: Features shown in this figure are for planning purposes and represent approximate locations. Engineering and/or survey accuracy is not implied.

SCAPPOOSE Oregon

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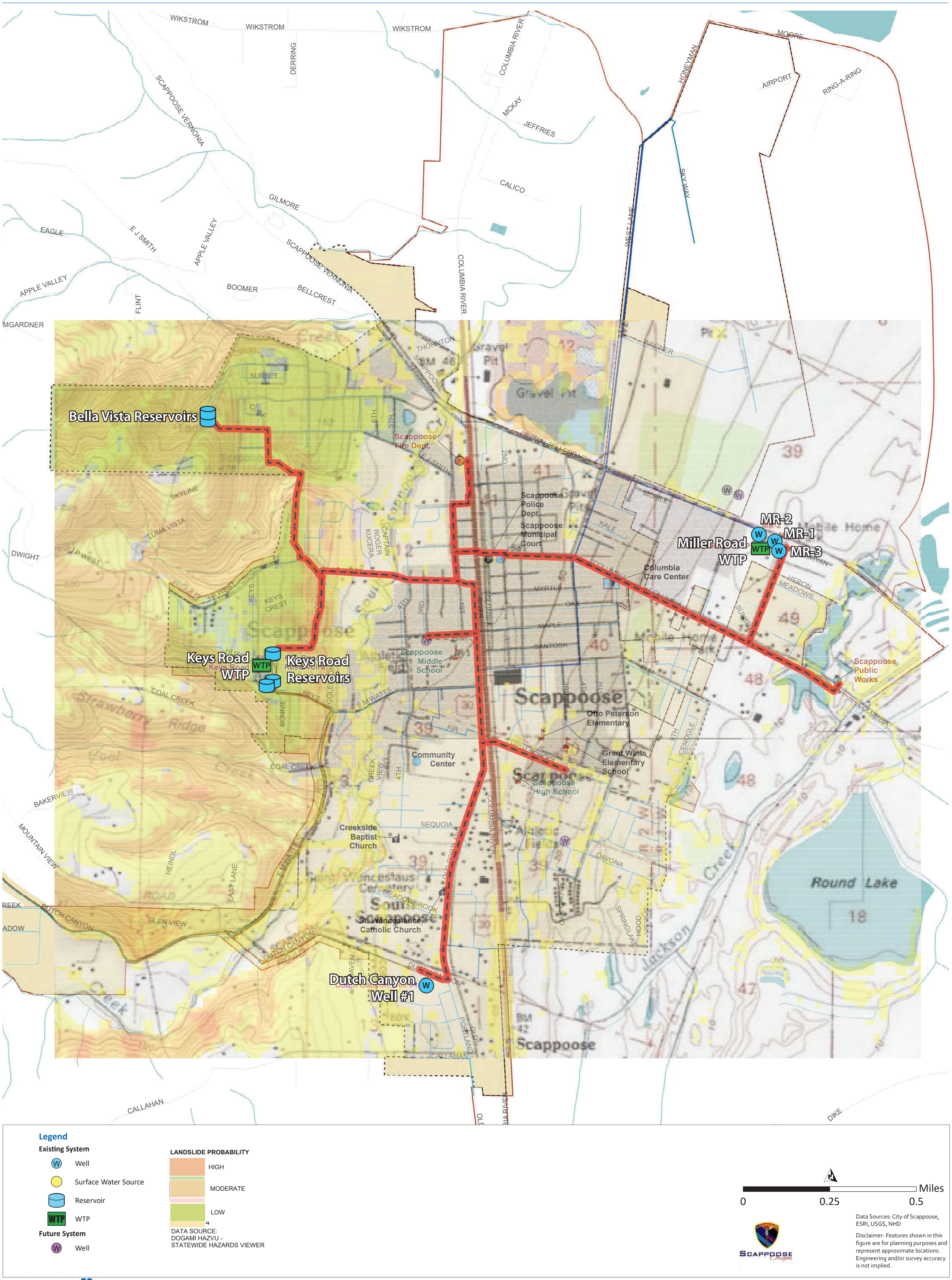
Figure 10.3: Seismic Backbone with Liquefaction Hazard



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V:\Client89\Scappoose\10738\0819\Fig10.4_Landslide_SeismicBackbone8.5x11.ai

Figure 10.4: Seismic Backbone with Landslide Hazard

The geohazard distribution is similar the hazards identified for the WTPs. The lower elevation region near Miller WTP and east of 4th Street has a low probability of liquefaction. The risk of liquefaction and lateral spread increases along Keys Crest near Keys WTP, then drops again at higher elevations. The probability of landslides is highest near the Bella Vista reservoir site and for the pipelines leading up the hill to the tanks.

Pipelines are susceptible to damage from two types of ground movement: permanent ground deformation (PGD) and transient ground deformation (TGD). Landslides, liquefaction, lateral spread, and faulting are PGD examples. Transient ground deformation is the rolling seismic wave passing through the region.

In general continuous pipelines such as welded steel pipe outperform segmented pipes. Welded steel pipe is perhaps the most common continuous pipeline material. Fusion welded HDPE pipe is another common continuous pipeline material. Restrained ductile iron (DI) pipe and Earthquake Resistant DI pipe are considered hybrid pipelines with continuous and segmented pipeline properties. Unrestrained DI pipe and bell and spigot polyvinyl chloride (PVC) are examples of segmented pipe used for waterlines. Anecdotal information gathered during informal interviews during the seismic assessment identify some of the City's newer pipelines as welded steel pipe. Record drawings or asset management data was not available to confirm this.

Identifying the pipeline materials is the next step to better understand the risk of damage to the backbone system during a seismic event. Continuous pipes will have a low probability of damage from TGD. Similarly continuous pipe is less susceptible to damage from PGD especially for smaller ground movements. A more detailed evaluation of permanent ground deformation hazards in areas with segmented pipelines will also help the City meet ORP performance objectives.

10.4 Ground Motion Parameters, and Evaluation Methodology

MJA's seismic hazards report included ground motion parameters for use in the WTP buildings seismic evaluation.

10.4.1 Seismic Hazard Ground Motion Parameters

The seismic vulnerability assessment considered a M9.0 scenario earthquake originating on the CSZ. For the M9.0 event, MJA conducted a seismic hazard evaluation to determine ground motion and other parameters applicable to the components assessed. The evaluation performed by MJA is set forth in the Seismic Hazard Evaluation Memorandum in Appendix A.

Seismic design parameters (Table 10.2) adopted for the structural evaluation of buildings were the ASCE 41 Basic Safety Earthquake 1 for Existing Buildings (BSE-1E). The BSE-1E has a 20 percent probability of exceedance in 50 years and 275 year mean recurrence interval. The ASCE 7-10 Design Basis Earthquake was adopted for water bearing structure's evaluation.

Table 10.2 Seismic Parameters

	Keys Road WTP	Miller Road WTP
S_S: Short-period spectral response acceleration parameter	0.970	0.960
S_1: spectral response acceleration parameter at a 1-s period	0.437	0.431
Site Class	E(1)	D1
S_{XS}: BSE-1 Design short-period spectral response acceleration parameter(2)	0.605	0.459
S_{X1}: BSE-1 Design spectral response acceleration parameter at a 1-s period(2)	0.392	0.265
S_{DS}: ASCE 7-10 Design short-period spectral response acceleration parameter(3)	0.605	0.714
S_{D1}: ASCE 7-10 Design spectral response acceleration parameter at a 1-s period(3)	0.700	0.451

Notes:
(1) Site Class assumed based on MJA hazards evaluation geotechnical information.
(2) ASCE 41-13 BSE-1E
(3) ASCE 7-10, ASCE 41-13 BSE-1N.

10.4.2 Seismic Evaluation Methodology

ASCE 41 classifies water treatment facilities as Risk Category IV. The Basic Performance Objectives for Existing Buildings (BPOE) adopted for this study in Immediate Occupancy (IO) structural performance and Position Retention Non-structural performance (1-B) as defined in ASCE 41.

10.4.2.1 Building Type Structures

The seismic structural evaluation of building-like structures was completed using the Tier 1 procedure of ASCE 41-13. This procedure uses a checklist-based approach to identify potential seismic structural deficiencies commonly observed in past earthquakes. The Tier 1 procedure also uses quick-check calculations to evaluate potential deficiencies in the primary components of the seismic load resisting system.

10.4.2.2 Liquid Containing Tanks and Structures

Liquid containing tanks and structures were evaluated for the load combination of gravity loads and seismic loads. The response spectrum analysis method was used to calculate the seismic loading on the structures.

The convective (sloshing) and impulsive forces (inertial) were calculated according to the recommendations of the American Concrete Institute (ACI) 350.3 for concrete tanks and American Water Works Association (AWWA) D100-11 for steel tanks. Modifications to these standards specified in the 2014 Oregon Structural Specialty Code were used along with the loads ASCE 41 BSE-1N earthquake and ASCE 7-10 importance factors for Risk Category IV structures.

Risk Category IV structures are defined in ASCE 7-10 as Buildings and other structures designated as essential facilities and structures required to maintain the functionality of other Risk Category IV structures.

10.4.2.3 Nonstructural

The seismic evaluation of the nonstructural components was performed using the ASCE 41-13 non-structural checklists and the TCLEE-22 checklists

10.5 Structural and Nonstructural Findings

10.5.1 Miller Road Water Treatment Plant

Miller Road WTP has a reinforced concrete filter building with three well sites. The record as-built drawings are dated 2004. The design was based on the 1997 Uniform Building Code and 1998 Oregon Structural Specialty Code.

10.5.1.1 Structural Findings

The filter building has reinforced masonry walls with flexible diaphragms for the occupied spaces. This structure is built on top of the cast-in-place concrete filters.

Miller Road 1 Emergency Well is in a wood framed building that predates the 2004 construction.

Miller Road 2 Production Well building is similar to the Filter Building with reinforced masonry walls and a flexible diaphragm.

Miller Road Well 3 has a small prefabricated weather enclosure.

All of the buildings at Miller Road WTP are on mat slab shallow foundations. The geotechnical hazard analysis for the Miller Road WTP site did not identify any ground deformation hazards.

Based on the building seismic design provisions the Filter Building and Well No. 2 are ASCE 41 benchmark buildings. As such, a structural seismic evaluation of the building structure is not required by ASCE 41 however evaluation of non-structural elements is still required. Benchmark buildings are assumed to meet the ASCE 41 life-safety performance objectives. Structural and non-structural evaluations were performed for the Miller Road WTP including benchmark buildings to evaluate the IO performance objective.

The Filter building walls and slabs use a variety of construction materials. The filters have cast-in-place concrete walls. The masonry walls between the filters are fully grouted masonry units. The masonry walls outside of the building core between the filters are partially grouted masonry units. The operations room floor slab is a cast-in-place concrete stiff diaphragm. The roof diaphragms are flexible metal decking diaphragms. The building does not fit neatly into an ASCE 41 building type classifications. It was checked using these structural checklists.

- 16.1 Basic Checklist.
- 16.1.2LS, IO Life Safety and IO Basic Configuration Checklist.
- 16.10LS, IO Building Types C2: Concrete Shear Walls with Stiff Diaphragms and C2A Concrete Shear Walls with Flexible Diaphragms.
- 16.15LS, IO Building Types RM1: Reinforced Masonry Bearing Walls with Flexible Diaphragms and RM2 Reinforced Masonry Bearing Walls with Stiff Diaphragms.

The checklist screening identified one potential structural deficiency (16.10LS, IO Diaphragm Continuity). The roof above the ground level chemical storage rooms is below the operations room floor forming a split level diaphragm. In this case the discontinuous diaphragm between Grid lines C and E appears to have a complete load path. The discontinuity could be mitigated with new vertical elements.

Due to this potential deficiency and the unusual combination of building materials and structural elements, detailed evaluation can determine to determine if the checklist screening missed any structural deficiencies in the Filter Building.

Well No. 1 building was evaluated as a light framed wood building using 16.6 LS, IO Building Type W1 Wood Light Frame Building Checklists. No structural seismic deficiencies were observed in the Miller Road 1 Emergency Well building.

Well No. 2 evaluation used the same Reinforced Masonry checklists as the Filter Building. No structural seismic deficiencies were observed in the Miller Road 2 Production Well building.

Well No. 3 enclosure is too small to classify as a building.

10.5.1.2 Nonstructural Findings

The following non-structural deficiencies were identified at Miller Road WTP using both the ASCE 41 Non-structural and the TCLEE-22 checklists.

Filter washwater troughs may not be designed to withstand seismic forces.

The large glass windows at the filter room may be damaged during an earthquake.

Soda ash storage tank is unanchored.

10.5.2 Dutch Canyon Well

The Dutch Canyon Well building is a partially grouted reinforced masonry building with wood decking and plywood diaphragm. The building has a mat slab shallow foundation. The building was constructed at the same time as the Keys Road WTP. The record as-built drawings are dated September 1979.

The building was checked using these structural checklists:

- 16.1 Basic Checklist.
- 16.1.2LS, IO Life Safety and IO Basic Configuration Checklist.
- 16.15LS, IO Building Types RM1: Reinforced Masonry Bearing Walls with Flexible Diaphragms.

10.5.2.1 Structural Findings

No seismic structural deficiencies were identified.

10.5.2.2 Nonstructural Findings

One non-structural deficiency was identified at Dutch Canyon Well using the ASCE 41 Non-structural and the TCLEE-22 checklists.

Some equipment anchorage is missing. Anchorage may be undersized at other locations.

The existing motor control and telemetry racks anchorage was not visible and could not be evaluated. Some of this equipment is currently being replaced.

The generator anchors are 3/8-inch diameter bolts. These may be undersized.

10.5.3 Keys Road Water Treatment Plant Site

The Keys Road WTP has numerous structures. The oldest structure is Keys Road Reservoir 3 which was built in 1947. Keys Road Reservoir 2 was built in 1967. Key Road Reservoir No. 1 is the newest tank built in 2004.

Keys Road Treatment Building and associated Washwater Basin and Pump Room were built during the 1979 project. In addition to these structures, there is a pump station at the southeast corner of the site built 1979.

The Keys Road WTP is not considered critical for water supply after a seismic event, where the Miller Road WTP provides sufficient supply immediately following a seismic event. However, the Reservoirs and Pump Station are critical and would be adversely impacted by the failure of the Key Road WTP during a seismic event. Therefore, the WTP was evaluated in this Chapter.

The geotechnical hazard analysis of the Keys Road WTP site found risk of liquefaction and lateral spreading. Only the 2.0 MG Reservoir foundation appears to consider these hazards in its design, though the efficacy of the tank's foundation is uncertain.

10.5.3.1 Keys Road Water Treatment Building

Keyes Road Water Treatment Building record drawings are dated September 1979. The building has precast concrete wall panels and columns. The roof is supported by timber trusses with a plywood diaphragm. The building houses two packaged filter units. A third cast-in-place concrete filter was attached to the original building in 2004.

The original building walls and filters are supported on a timber pile foundation. Filter 3 is supported on auger cast piles.

The Filter 3 drawings were not available for review.

The building was checked using these structural checklists.

- 16.1 Basic Checklist.
- 16.1.2LS, IO Life Safety and IO Basic Configuration Checklist.
- 16.12LS, IO Building Types PC1: Precast or Tilt-Up Concrete Shear Walls with Flexible Diaphragms.

Structural Findings

Numerous structural deficiencies were found at the water treatment building including geometric irregularities, and a soft story created by the large glass windows and openings at the building's main entrance.

Review of the record drawings and the walk-through identified a severe lack of seismic anchorage and restraint at the filter units and the building itself. The precast building panels and columns have no record of significant lateral restraint. Filters 1 and 2 are not anchored to the foundation.

Nonstructural Findings

The chemical storage tanks appear to be anchored, however a concrete curb obscures the tank to floor connection. The piping is generally braced.

10.5.3.2 Keys Road Washwater Basin

The Washwater basin is a below grade, open top, reinforced concrete tank. The Pump room is an enclosed reinforced concrete structure adjacent to the basin. A quick check of the concrete walls, similar to the methodology adopted by ASCE 41, found the walls have insufficient capacity to resist the design loads specified by the current building code. The tank baffles are likely to be damaged by sloshing contents during an earthquake.

The pumps and piping appear to have adequate seismic restraint.

10.5.3.3 Keys Road Pump Station

Record drawings of the pump station at the southeast corner of the WTP site were not available. Visual assessment of the brick building are not conclusive since masonry reinforcing could not be confirmed. Non-destructive testing is required to assess the building's lateral force resisting system and its ability to withstand the anticipated seismic shaking.

The pumps in the building have an unusual thrust restraint and anchor system. There appears to be a complete load path but its capacity is uncertain.

10.5.3.4 Keys Road Reservoir No. 1

Reservoir No. 1 is a 2,000,000 gallon 123 foot diameter pre-stressed concrete tank with a 24-foot tall wall. The tank has a mat slab foundation supported on stone columns. The lower quarter of the wall is below grade. The tank was built in 2003.

Structural Findings

The tank exterior condition is good with no significant cracks or visible corrosion stains.

The record as-built drawings have general configuration information. Stone column foundation information and pre-stressing details are missing as this was appears to have been procured as contractor designed construction. The missing information would be in shop drawing submittals.

Seismic evaluation was limited to freeboard calculations. The required freeboard is slightly larger than distance from overflow elevation to the underside of the tank roof low point. The aluminum roof may be damaged by the sloshing wave impact.

The ASCE 41 benchmark building list does not include tanks, however pre-stressed concrete tanks have led the tank industry in seismic design. It is likely that the tank has a well-detailed lateral force resisting system with reliable seismic performance. Review of the detailed as-built drawings is necessary to confirm this.

10.5.3.5 Keys Road Reservoir No. 2

Reservoir No. 2 is a 1,000,000 gallon 84 foot inside diameter pre-stressed concrete tank with a 25.5-foot tall wall. The tank has a mat slab foundation. Grade varies around the tank perimeter. About half of the uphill portion of the tank wall is below grade. About a quarter of the tank is buried on the uphill side. The tank was built in 1967.

Structural Findings

The tank exterior condition is good with no significant cracks or visible corrosion stains.

The record drawings have general configuration information and limited pre-stressing details. The missing information would be in shop drawing submittals. Three tank alternatives are shown on the record drawings. The record drawings appear to be contract bid documents and are not as-built drawings. Visual assessment indicates that a pre-stressed tank was built. Detailed site assessment with non-destructive or possibly destructive testing would be required to confirm this assumption.

The tank is partially embedded in the hillside which may result in downhill movement during an earthquake especially if lateral spread at the site were to occur.

Seismic evaluation was limited to freeboard calculations. The required freeboard is larger than distance from overflow elevation to the underside of the tank roof low point. The uplift pressure from the sloshing wave impact exceeds the roof weight.

This tank appears to be an early example of a strand wrapped, pre-stressed concrete tank. The details on the contract documents indicate that seismic restraint is provided at the base of the tank wall. The roof to wall connection is similar to current detailing practice. If a pre-stressed tank similar to the record drawings was constructed, it is likely that the tank has a well-detailed lateral force resisting system with reliable seismic performance, though designed to a seismic load significantly less than currently design earthquake.

10.5.3.6 Keys Road Reservoir No. 3

Reservoir No. 3 is a 200,000 gallon, 52.5-foot inside diameter conventionally reinforced concrete tank with a 19-foot tall wall. The tank has a mat slab foundation. The lower third of the wall is below grade. The tank was built in 1946 and upgraded in 1999. Since that time or shortly afterward the tank has been out of service.

The tank exterior condition is good with no significant cracks or visible corrosion stains.

The record drawings are incomplete. A seismic evaluation was not performed due to lack of information including operating water surface elevation.

10.5.4 Bella Vista Reservoirs

The Bella Vista Reservoir site has two welded steel tanks: 0.30 MG tank built in 1968 and a 0.37 MG tank built in 2003. The contract drawings for both tanks were available for review. These drawings provide general configuration information. Detailed drawings were not available as the steel tank design appears to have been performed by the contractor's engineer.

The tanks appear to be on reinforced concrete ring wall foundations. The geotechnical hazard analysis for the Bella Vista Reservoir site identified potential landslide risk.

10.5.4.1 Bella Vista 0.30 MG Reservoir

Analysis of the 0.30 MG reservoir found that the tank should be mechanically anchored. The tank record drawings show the tank shell is set in a concrete curb and is self-anchored. The site walk down confirms this condition though foundation details are not clear. Self-anchored tanks are susceptible to elephant's foot buckling which can result in catastrophic loss of tank contents.

The tank freeboard is insufficient. Sloshing damage at the top of the tank is likely during an earthquake.

The tank inlet and outlet are through the bottom of the tank shell. This type of connection is less susceptible to seismic damage than connections through the tank shell. However, the record drawings show the overflow and drain pipes are close to the tank shell. This makes them more susceptible to damage as the unanchored tank rocks.

10.5.4.2 Bella Vista 0.37 MG Reservoir.

Analysis of the 0.37 MG reservoir found that the tank should be mechanically anchored. The site walk down confirmed the tank is anchored at 16 anchor bolt chairs. Based on preliminary analysis the anchor bolts may be undersized.

The tank freeboard is nearly equal to the expected sloshing wave height. Sloshing contents may damage the roof rafters during an earthquake.

The tank inlet and outlet are through the bottom of the tank shell. This type of connection is less susceptible to seismic damage than connections through the tank shell. The record drawings show the overflow and drain pipes are sufficiently away from the tank shell to minimize risk of seismic damage.

10.5.5 Glenn View Booster Station

The Glenn View Booster Pump station is a small package pump station installed in 2010. The pump station equipment is housed in a skid-mounted Container. The skid is anchored to a concrete foundation slab and the pumps and piping are braced. No seismic deficiencies were identified.

10.6 Recommendations

The following section summarizes recommendation to address potential deficiencies identified in the previous sections.

10.6.1 Miller Road Water Treatment Plant

There were no significant deficiencies identified at the Miller Road WTP. The Filter building passes the ASCE 41 screening checklists, however a detailed seismic evaluation might identify potential deficiencies resulting from the mix of reinforced concrete and masonry construction.

A plant wide non-structural seismic upgrade will address any unanchored equipment and stored material concerns.

10.6.2 Dutch Canyon Well

There were no significant deficiencies identified at the Dutch Canyon Well. Equipment anchorage should be evaluated as part of ongoing and future maintenance activities.

10.6.2.1 Dutch Canyon Well Transmission Main

The Dutch Canyon Well supply is treated for aesthetic reasons (high levels of iron) at Keys Road WTP Filter No. 3. The City intends to supply water directly from the well in the event that the transmission main fails in a seismic event. Therefore, the transmission main was not considered part of the backbone and not evaluated as part of this study.

10.6.3 Keys Road Water Treatment Plant Site

Keys Road WTP Site presents the greatest risk of catastrophic failure during a CSZ event. The Geotechnical Hazard TM found two potential geohazards at this site. Potential liquefaction and lateral spreading may compromise any structure at the site.

A site-specific geotechnical evaluation is needed to assess the liquefaction and spreading magnitude and the ability of the building foundations to withstand the anticipated ground movement.

10.6.3.1 Keys Road Water Treatment Building

Significant structural and nonstructural deficiencies were identified in the Treatment Building. In general terms, there are two potential failures that would lead to catastrophic failure leaving the plant inoperable for a long period of time:

- Filters 1 and 2 are not anchored. They can slide off of their concrete bases with significant and possibly irreparable damage.
- The 1979 building housing Filters 1 and 2 and the chemical storage and feed system does not have a reliable lateral force resisting system. This could result in collapse precast panels or the entire building.

Following the geotechnical evaluation, a detailed plant-wide assessment resulting in plans to seismically upgrade the entire facility is a first step towards remediating the structural and non-structural deficiencies.

A detailed evaluation of the building and treatment process will determine a cost effective approach to retrofit for life safety. Filter 3 (treats Dutch Canyon well water) should be decoupled from the treatment building. It can be retrofit as a free standing structure, where retrofit feasibility is predicated on favorable results from the geotechnical and foundation evaluation. If the WTP cannot be retrofitted, it should be isolated from the reservoirs and booster pump station to aid in operation of those structures after a seismic event.

10.6.3.2 Keys Road Washwater Basin

The Washwater Basin is a conventionally reinforced concrete tank that is embedded in the surrounding hillside. The tank wall reinforcing is adequate for the seismic loads however the floor slab reinforcing is undersized for the wall bending moments transmitted to it. This condition could allow the walls to rotate at the base potentially resulting in leaks. There is adequate shear capacity which indicates a sudden catastrophic failure is unlikely.

The basin may be at risk of sliding out from the hillside due to the unbalanced soil load, as is the adjacent pump room.

Conducting a detailed seismic evaluation and developing a structural retrofit to strengthen the floor slab will improve the Washwater Basin's seismic reliability. The site specific geotechnical evaluation should include global stability for the unbalanced soil loads. Retrofit feasibility is predicated on favorable results from the geotechnical and foundation evaluation.

10.6.3.3 Keys Road Pump Station

Because little is known about the Keys Road Pump Station building a seismic evaluation was not completed. Perform testing to determine if the building walls are reinforced, and expose the roof to wall connections to evaluate their capacity. Conduct a seismic evaluation of the building using this information.

10.6.3.4 Keys Road Reservoir No. 1

Evaluate the tank's global stability using the site specific geohazards evaluation findings to determine if a foundation retrofit is required to meet the ORP performance objectives.

10.6.3.5 Keys Road Reservoir No. 2

Evaluate the tank's global stability using the site specific geohazards evaluation findings to determine if a foundation retrofit is required to meet the ORP performance objectives.

10.6.3.6 Keys Road Reservoir No. 3

Keys Road Reservoir No. 3 is not considered a critical facility.

10.6.4 Bella Vista Reservoirs

This screening level evaluation identified two potential seismic hazards at the site, landslide and tank shell damage due to rocking. Insufficient information was available to fully assess these potential hazards. Detailed evaluation along with a consequence of failure will help better understand the risks at this site. The detailed evaluation effort would include the following:

- Assess landslide potential with a site specific geotechnical evaluation.
- Confirmation of foundation dimensions by trenching or other means if construction records are not available.
- Perform non-destructive testing of tank shell thickness and a desktop evaluation to evaluate buckling failure risk at each tank.

10.6.4.1 Bella Vista Reservoir No. 1

This 0.30 MG reservoir is unanchored with pipe inlet and outlet pipes passing through the floor but close to the tank shell. As such this tank presents an increased risk of damage during an earthquake.

Based on the screening level evaluation the Bella Vista No. 1 requires a foundation seismic retrofit to anchor the tank. Detailed evaluation may identify additional modifications such as tank shell lower plate strengthening.

Obtain shop drawings or conduct non-destructive testing to as-built the tank shell and perform detailed seismic evaluation using the current building code to evaluate the tank's seismic capacity.

Develop and implement a seismic retrofit including anchoring the tank and a foundation retrofit sized for the anchor bolt uplift force.

10.6.4.2 Bella Vista Reservoir No. 2

This 0.37 MG reservoir is mechanically anchored with pipe inlet and outlet pipes through the floor and away from the tank shell. As such it should perform well during an earthquake, though the anchor bolts may be undersized.

Obtain shop drawings or conduct non-destructive testing to as-built the tank shell and perform detailed seismic evaluation using the current building code to evaluate the tank's seismic capacity.

If deemed necessary by the detailed evaluation, develop and implement seismic retrofit.

10.6.4.3 Glenn View Booster Pump Station

The skid mounted booster pump station is anchored and should perform well during an earthquake. No modification are recommended.

10.7 50-Year Mitigation Plan

Up to this point, this chapter:

- Identified the seismic hazards within the City's system.
- Detailed the seismic system that will supply water after the CSZ earthquake.

- Evaluated the anticipated performance of the critical facilities in the system.
- Recommended actions for the City to begin planning for mitigating expected damage.

The scope of these improvements is vast, and they are intended to be accomplished over the next 50 years. Chapter 8 outlines what seismic projects the City plans to complete in the next 20 years and where the recommendations outlined in this section should be implemented over the next 50 years.

Table 10.3 shows the City's 50-year schedule for conducting additional evaluations and implementing improvement recommendations. The table is broken out by the critical facilities that were identified by the City, and the projects were given a 10-year time range for when they plan to be completed.

As reflected in its 20-year CIP, the City plan to complete seismic mitigation on select critical facilities:

- Miller Road WTP repair and replacement including minor Seismic Retrofit will mitigate deficiencies identified in this Chapter.
- Keys Road WTP Seismic and Life Safety Audit is planned. Given the potential for catastrophic failure of the WTP, the City plans to conduct an audit to determine if deficiencies identified in this Chapter can be cost effectively mitigated.
- Reservoir Seismic Retrofits. In the next 20 years, seismic retrofits to mitigate current deficiencies are planned for the Keys Road 1.0 MG reservoir and, if necessary, the Keys Road 2.0 MG reservoir and Bella Vista 2 Reservoir.
- Seismic Backbone Piping. No specific projects are planned to complete entire sections of the backbone for the next 20 years. However, any pipe installed along the backbone (due to development, condition related replacement, etc.) will be seismic resilient piping.

Mitigation of the remaining critical infrastructure will be addressed outside of the 20-year CIP planning period.

Table 10.3 Preliminary Mitigation Plan Schedule

Project	2019 – 2028	2029 – 2038	2039 – 2048	2049 – 2058	2059 – 2068
Water Treatment Plants					
Keys Road WTP	X				
Miller Road WTP	X				
Reservoirs					
Keys Road 1, 2 MG Reservoir		X			
Keys Road 2, 1 MG Reservoir		X			
Keys Road 3, 0.2 MG Reservoir					
Bella Vista 1, 0.30 MG				X	
Bella Vista 2, 0.37 MG		X			
Pump Stations					
Keys Road			X		
Glen View					X
Wells					
Miller Road 1 Emergency Well					X
Miller Road 2 Production Well					X
Miller Road 3					X
Dutch Canyon	X				
Seismic Backbone Piping					
Section A			X		
Section B				X	
Section C				X	
Section D					X
Section E					X

10.8 Executive Summary

This chapter provided the City with a risk assessment and 50-year mitigation plan that satisfies the OHA requirements found in OAR 333-061-0060, which includes a seismic backbone map of the City's critical facilities, a risk assessment of those critical facilities, and a 50-year mitigation plan that provides a general timeline of when the recommended seismic improvements would be implemented. General recommendations and findings are as follows:

1. **Identified Seismic Backbone:** The City has identified a seismic "backbone" that, when completed, will connects emergency service locations (fire department and shelters) to seismically resilient water infrastructure (Miller Road WTP and water reservoirs). New seismically resilient piping is needed to create the "backbone".
2. **Seismic Retrofit of Existing Facilities.** Seismic retrofits are needed to bring existing Facilities - which likely were designed to withstand a smaller seismic events - to current standards. Much of the City's existing facilities may require relatively minor retrofits, including: Miller Road WTP and wells, Dutch Canyon Wells, 2 MG Keys Road Reservoir, Belle Vista Reservoir 2, and Glenn View Pump Station. More extensive retrofits may be required for 1 MG Keys Road Reservoir, Keys Road Booster Pump Station, and Belle Vista Reservoir 1.
3. **Potential Catastrophic Failure of Keys Road WTP:** The Keys Road WTP has the potential to catastrophically fail during a CSZ earthquake. A Seismic and Life Safety Audit is planned to identify potential means of addressing this serious concern.

Photo 1: Overview of Miller Road WTP Buildings.

Photo 2:
Rear view of
Dutch Canyon
Pump Station.

Photo 3:
Keys Road
Treatment Building
main entrance
complex
configuration with
large openings.

Photo 4:
Filter 3 is the exposed masonry block room and cast-in-place tank Treatment Building addition at Keys Road.

Photo 5:
Exposed aggregate precast concrete panels and columns anchorage is nominal and unable to withstand seismic shaking without significant damage at Keys Road.

Photo 6:
Keys Road Filters are not anchored to concrete foundation block.

Photo 7:
Rigid piping connections between filter process areas will be damaged if the filters shift during an earthquake at Keys Road.

Photo 8:
Keys Road Chemical Storage tank anchors are obscured by concrete curb making size and capacity evaluation difficult.

Photo 9:
Unanchored chemical metering pumps are likely to experience damage during an earthquake at Keys Road.

Photo 10:
Keys Road Pump Station, record drawings were not available for review.

Photo 11:
Piping in the Keys Road Pump Station has unusual thrust and lateral restraint systems.

Photo 12: Keys Road Reservoir 1 aluminum roof may experience damage from sloshing contents

Photo 13: Bella Vista Reservoir 1 is not anchored.

Photo 14:
Curb at base of Bella Vista Reservoir 1 shell plate.

Photo 15:
Bella Vista Reservoir 2 has anchors that may be undersized.

Appendix A
DEMOGRAPHIC AND DEMAND

Oregon Health Authority

City of Scappoose

Water System Survey
OHA Drinking Water Services

PWS ID: 41 **00792**
Survey Date: **11/3/16**

Deficiency Summary

Surveyor: James Nusrala

Date Corrective Action Plan is due: January 16, 2017 **County:** Columbia

Yes	No	Significant Deficiencies and Rule Violations:	Date to be corrected	Date corrected
☒	☐	**Source:**		
		Well construction: Dutch Canyon well vent not properly screened.		
		Spring/other source: No significant deficiencies or rule violations noted.	NA	
☐	☒	**Treatment:**		
		Surface water treatment: No significant deficiencies or rule violations noted.	NA	
		Disinfection: No significant deficiencies or rule violations noted.	NA	
		Other treatment: No significant deficiencies or rule violations noted.	NA	
☐	☒	**Finished Water Storage:** No significant deficiencies or rule violations noted.	NA	
☐	☒	**Distribution:** No significant deficiencies or rule violations noted.	NA	
☐	☒	**Monitoring:** No significant deficiencies or rule violations noted.	NA	
☐	☒	**Management & Operations:** No significant deficiencies or rule violations noted.	NA	
☒	☐	**Operator Certification:** No written under certified operator protocol for distribution.		
☒	☐	**Other Rule Violations:** Need to calculate and report amount of fluoride added to water and level of fluoride in water – OAR 333-061-0085(3)(d).		

Comments:
See letter dated November 28, 2016 for details and comments and recommendations.

☒ **Source Deficiencies:**

Well Construction Deficiencies:

- ☐⊕ Sanitary seal and casing not watertight
- ☐⊕ Does not meet setbacks from hazards
- ☐⊕ Wellhead not protected from flooding
- ☐⊕ No raw water sample tap
- ☐⊕ No treated sample tap (if applicable)
- ☒⊕ No screen on existing well vent

Spring Source Deficiencies:

- ☐⊕ Springbox not impervious durable material
- ☐⊕ No watertight access hatch/entry
- ☐⊕ No screened overflow
- ☐⊕ Does not meet setbacks from hazards
- ☐⊕ No raw water sample tap
- ☐⊕ No treated sample tap (if applicable)

☐ **Treatment Deficiencies/Violations:**

Surface Water Treatment Deficiencies:

- ☐+ Turbidity standards not met - *0030(3)*
- ☐+ Turbidimeters not calibrated per manufacturer or at least quarterly - *0036(5)(b)(A)(ii)*
- ☐⊕ Incorrect location for compliance turbidity monitoring
- ☐⊕ If serving > 3,300 people no alarm or auto plant shut off for low chlorine residual
- ☐⊕ For conventional or direct filtration: No alarm or plant shut off for high turbidity
- ☐⊕ For conventional filtration: Settled water not measured daily
- ☐⊕ For conventional or direct filtration: Turbidity profile not conducted on individual filters at least quarterly
- ☐⊕ For cartridge filtration: No pressure gauges before and after cartridge filter
- ☐⊕ For cartridge filtration: Filters not changed according to manufacturer's recommended pressure differential
- ☐⊕ For diatomaceous earth filtration: Body feed not added with influent flow
- ☐+ For membrane filtration: Turbidimeter not present on each unit - *0050(4)(c)(G)*
- ☐+ For membrane filtration: Direct integrity testing not done at least daily - *0036(5)(b)(F)*

Disinfection Deficiencies/Violations:

- ☐+ DPD or EPA approved method not used - *0036(9)(d)*
- ☐+ Free chlorine residual not maintained - *0032(3/5)*
- ☐+ Chlorine not measured & recorded as required - *0036(9)*
- ☐+ Minimum CT requirement not met all times - *0032(3/5)*
- ☐⊕ No means to adequately determine flow rate on contact chamber effluent line
- ☐+ pH, Temperature, and chlorine residual not measured daily at first user - *0036(5)(a/b)*
- ☐⊕ Failure to calculate CT values correctly
- ☐⊕ No means to adequately determine disinfection contact time under peak flow and minimum storage conditions

UV Disinfection Violations (OAR 333-0050(5)(k)):

- ☐+ Bypass around UV system
- ☐+ Lamp sleeve not cleaned
- ☐+ Lamp not replaced per manufacturer
- ☐+ No intensity sensor with alarm or shut-off

Other Treatment Violations:

- ☐+ Non-NSF approved chemicals - *0087(6)*
- ☐+ Corrosion control parameters not met - *0034*

☐ **Distribution System Violations:**

- ☐+ System pressure < 20 psi - *0025(7)*

Cross Connection (OAR 333-061-0070):

- ☐+ No ordinance or enabling authority (CWS)
- ☐+ Annual Summary Report not issued (CWS)
- ☐+ Testing records not current (CWS, NTNC, TNC)
- ☐+ No Cross Connection Control Specialist (CWS ≥ 300 connections)

☐ **Finished Water Storage Deficiencies:**

- ☐⊕ Hatch not locked or adequately secured
- ☐⊕ Roof and access hatch not watertight
- ☐⊕ No flap valve, screen, or equivalent on drain
- ☐⊕ No screened vent

☐ **Monitoring Violations:**

- ☐+ Monitoring not current - *0025(1)*
- ☐+ Unaddressed MCL violations or LCR AL exceedances - *0030*
- ☐+ No Coliform Sampling Plan - *0036(6)(a)(G)*

☐ **Management & Operations Violations:**

- ☐+ No operations and maintenance manual - *0065(4)*
- ☐+ Emergency response plan not completed - *0064(1)*
- ☐+ Major modifications not approved (plan review) - *0050*
- ☐+ Master plan not current (≥ 300 con.) - *0060(5)*
- ☐+ Annual CCR not submitted (CWS) - *0043(1)(a)*
- ☐+ PNC or out of compliance with AO
- ☐+ Public notice not issued as required - *0042*

☒ **Operator Certification Violations:**

- ☐+ No certified operator at required level - *0065(2)*
- ☒+ No protocol for under certified operator - *0225(2)*

☒ **Other Rule Violations:** OAR 333-061-0085(3)(d) – report daily amount of fluoride added and fluoride levels to DWS.

⊕ Significant deficiency per OAR 333-061-0076
+ Rule violation per OAR 333-061-XXX

Inventory and Narrative

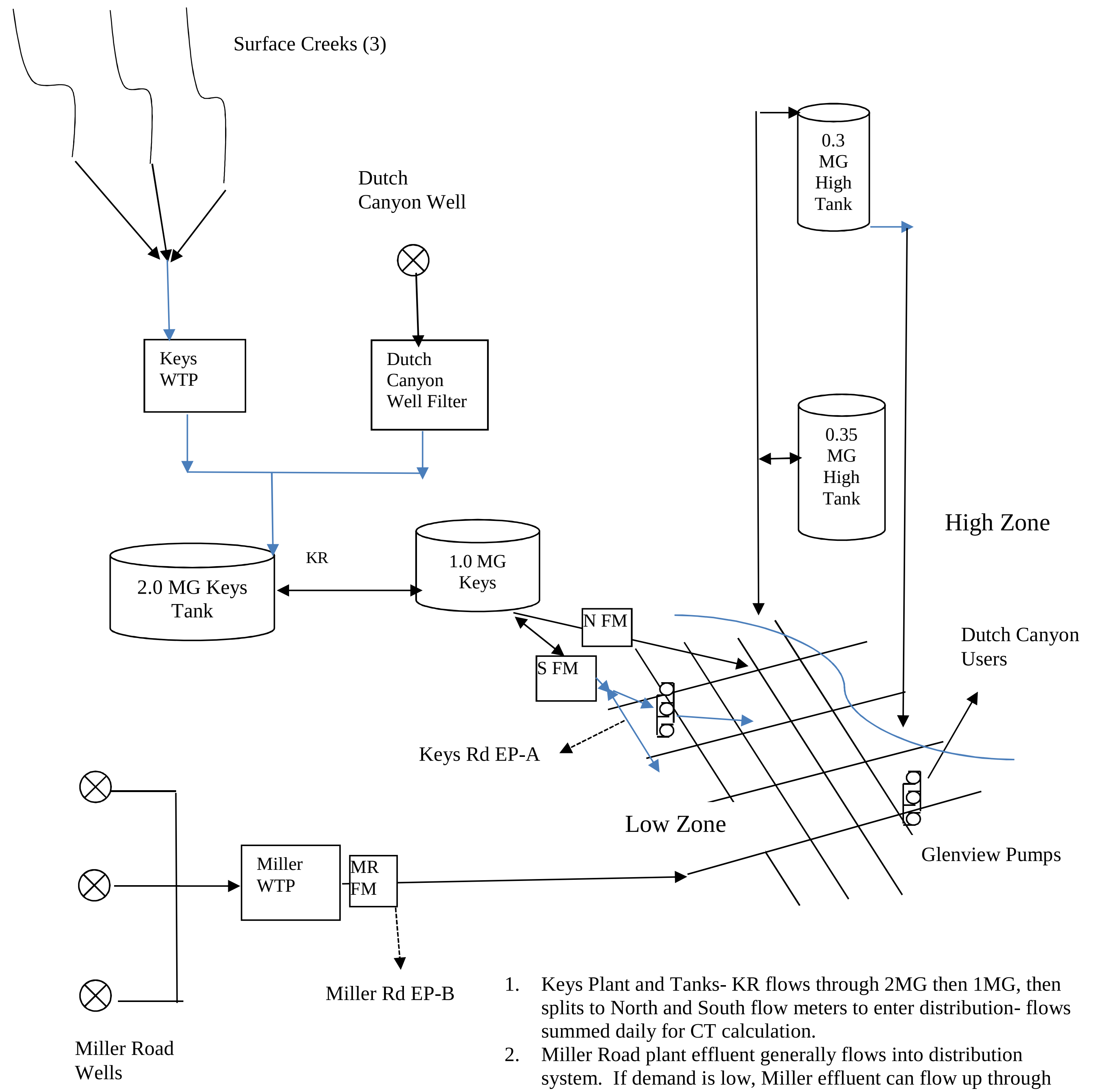
☐ Outstanding Performer					
Type:	Community (C)	**Status**	**Size**	**Season:**	Year-round
License:	Not Licensed	Population:	6,800	Begins: (mm/dd)	1/1
Responsible Agency:	State	Connections:	2,479	Ends: (mm/dd)	12/31
Service Characteristics:	Residential: City or Town (MU)				
Ownership:	4 - Local Government				
Operator Certification Requirements:	WD: 2	WT: 2	FE ☒ Small WS ☐		

Primary Administrative Contact (Mailing Address):			
Contact Name:	Darryl Sykes	Phone:	(503) 543-7185 or 543-5894; ext 6-MR; 7-KR WTPs
Title:	Water Plant Supervisor	Cell:	(503) 369-0297
Street Address:	33568 E. Columbia Ave	Emergency #:	(503) 369-0297
City/State/Zip:	Scappoose, OR 97056	Email:	darrylsykes@ci.scappoose.or.us
Legal/Owner Address:			
Contact Name:	City Hall	Phone:	( )
Title:		Cell:	( )
Street Address:	33568 East Columbia Avenue	Emergency #:	( )
City/State/Zip:	Scappoose, OR 97056	Email:	
System Physical Address:			
Contact Name:	Miller Road WTP / Keys Road WTP	Phone:	( )
Title:		Cell:	( )
Street Address:	52515 NE Miller Rd / 52212 Keys Rd	Emergency #:	( )
City/State/Zip:	Scappoose, OR 97056	Email:	
Emergency Systems Available:			
Name:	N/A	**PWS ID#:**	**41**

Narrative:

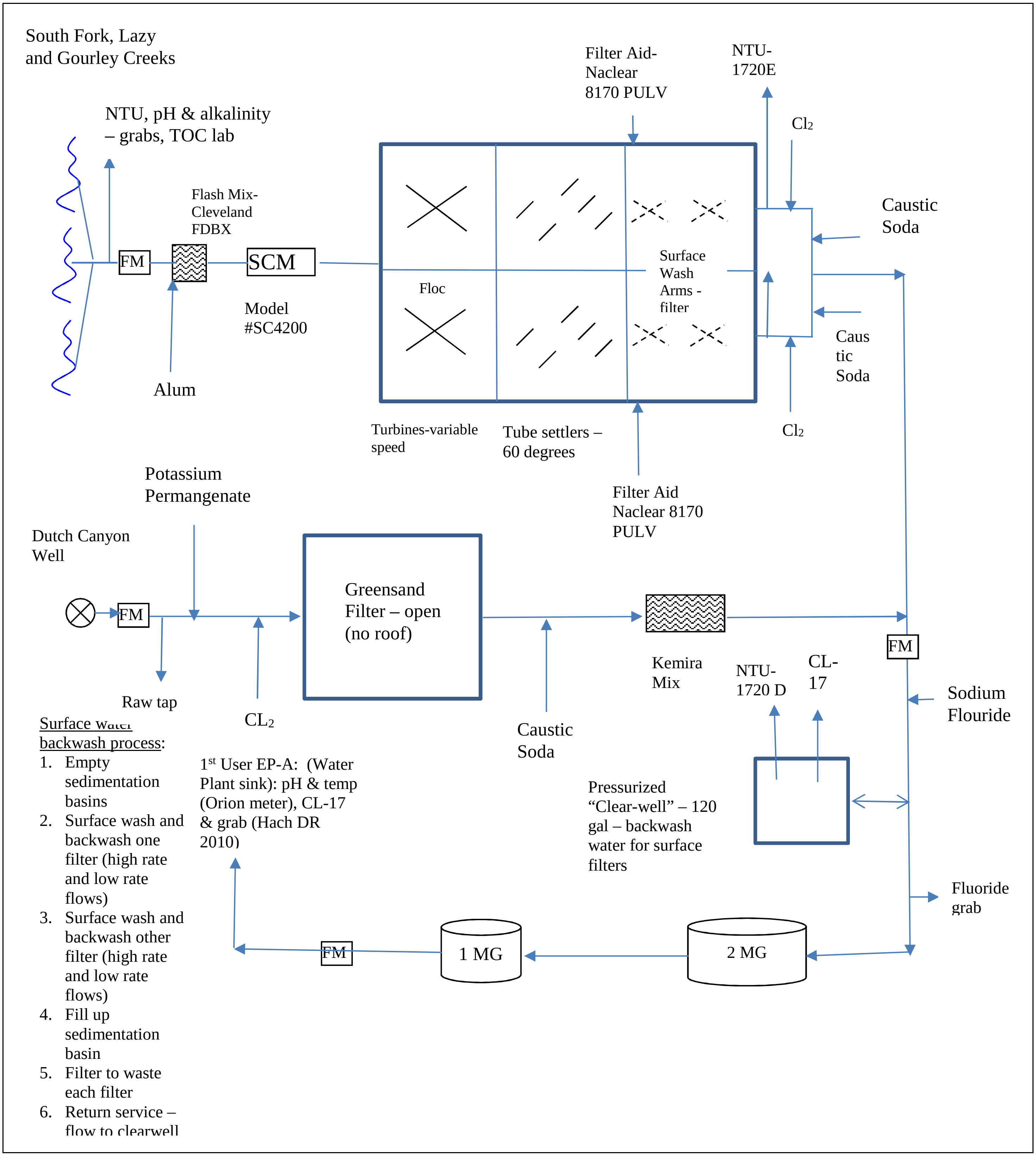
The City of Scappoose operates two separate treatment plants. A surface water source from three creek intakes is treated by a 2.0-log *giarda* removal conventional filtration package plant with tube settlers, and a groundwater source (Dutch Canyon well) by Greensand filtration for iron and manganese removal, at the Keys Road facility. Three wells are treated separately with Greensand filtration at the Miller Road facility. The primary sources are the Miller Road wells #1, #2 and #3, the Dutch Canyon well, and the Keys Road sources. All sources are fluoridated and chlorinated. Contact time is achieved for the Keys Road treatment system through two separate storage tanks; water flows first through a 2.0 Million Gallon-MG tank, and then a 1.0 MG tank. Water from the Miller Road plant can also fill the 1.0 MG Keys Rd tank in addition to directly serving the distribution, if the demand is low. A tracer study has been completed for the 2.0 MG storage tank, and a separate tracer study for the 1.0 MG storage tank, and the system calculates contact time according to a July 25, 2011 final approval for the 1 MG tracer study. Chlorination is practiced for the Miller Road sources to assist with the iron and manganese removal, not for contact time. Chlorination is also used to assist with iron and manganese removal for the Dutch Canyon well at the Keys Road facility. Corrosion control is performed using both caustic soda (Keys Rd. facility), and soda ash (Miller Rd. facility). The distribution system consists of four storage reservoirs with a combined volume of 3.65 MG. A fifth reservoir, of 0.3 MG is located at the Keys Rd facility but not currently used.

General Water System Schematic

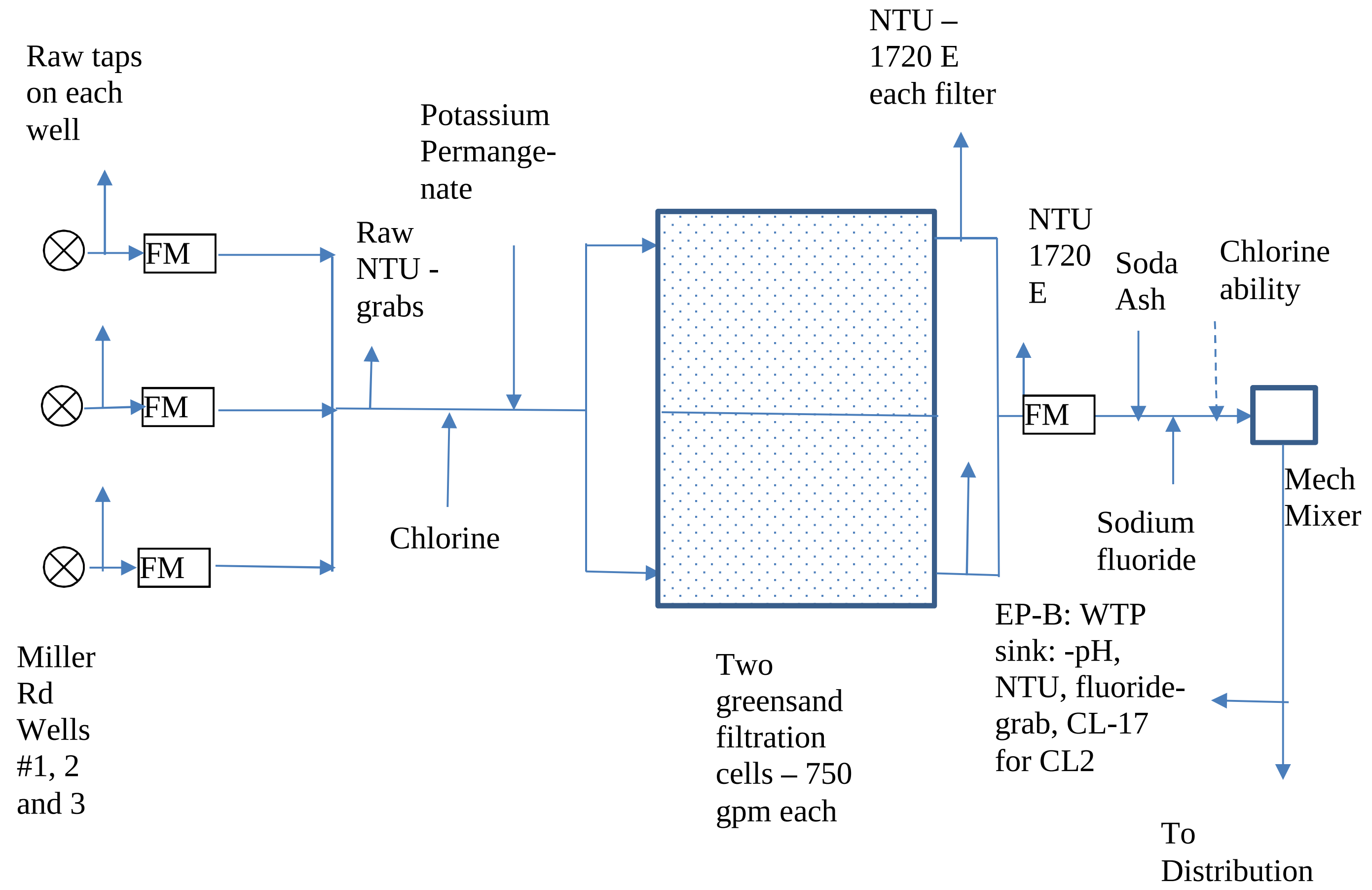


1. Keys Plant and Tanks- KR flows through 2MG then 1MG, then splits to North and South flow meters to enter distribution- flows summed daily for CT calculation.
2. Miller Road plant effluent generally flows into distribution system. If demand is low, Miller effluent can flow up through South flow meter to feed 1 MG Keys tank.
3. Total system daily production = MR flow + N Keys flow + S Keys flow (forward) – S Keys flow (reverse).
4. Keys 1 and 2 MG tank depth levels are the same as the piping between them keeps them equalized.

Keys Road WTP Schematic

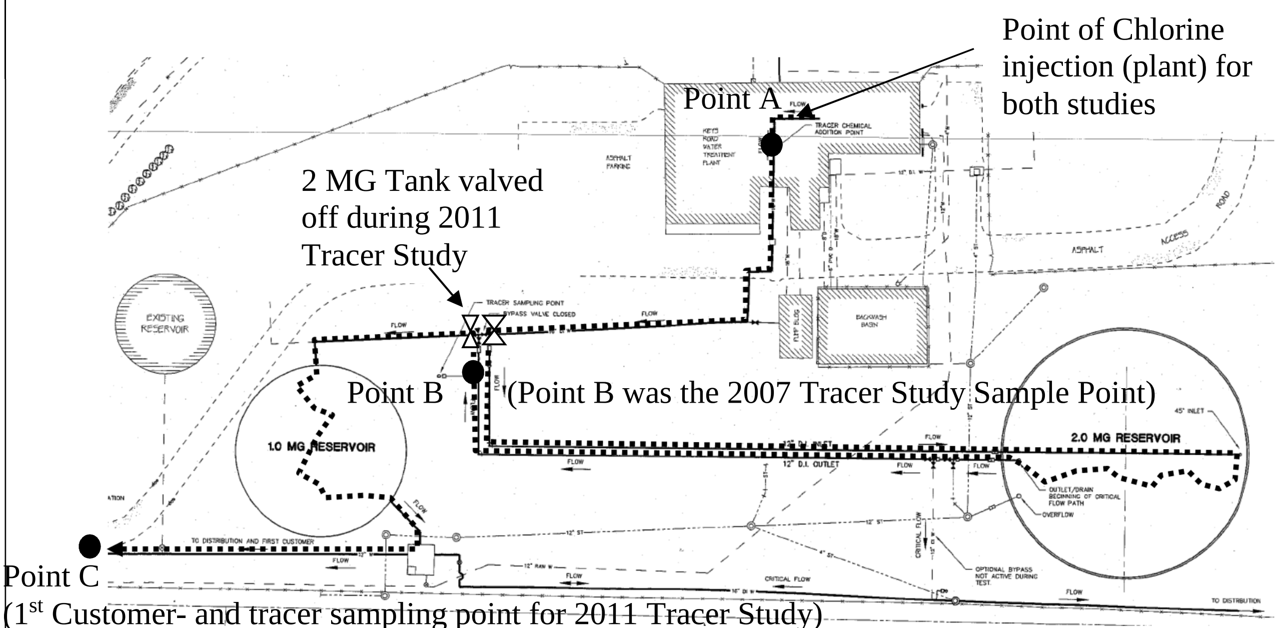


Miller Road WTP Schematic



2007 and 2011 Tracer Studies Summary

Tracer Study Schematic (from 2007 Tracer Study)



2007 Tracer Study – measured contact time from Point A to Point B (leaving 2MG reservoir):

- Contact Time = Sum of the following [from December 9, 2008 final approval letter for 2007 study]
  1. Segment 1: Time (from Pt A to Pt B) [piping and 2 MG reservoir] – measured in study empirically
  2. Segment 2: Time (from Pt B to Pt C) [piping only] – calculated using plug flow
  3. Segment 3: Time through 1 MG [tank only] – calculated using estimated baffling factor
- Contact Time = (-0.028 Q + 75) [segment 1] + (2,094.03/Q) [segment 2] + (33,333 / Q) [segment 3]
- CT achieved = C x total contact time above; where C is chlorine residual measured in mg/l at 1st user (Point C)

Note: December 2008 final approval letter from DWP noted that a tracer study needed on 1 MG tank to replace the estimated baffling factor for the 1MG tank.

2011 Tracer Study – measured contact time from Point A to Point C (excluding the segment to and from 2 MG reservoir, as this section was valved off to isolate 1 MG reservoir and associated piping)
Baffling factor of 1 MG reservoir determined by subtracting out the associated pipe volume used during the study from total contact time measured.

- Contact Time = Sum of the following [from July 25, 2011 final approval letter for 2011 study]-both tanks used
  1. Segment 1: Time (from Pt A to Pt B) [piping and 2 MG reservoir] – measured in 2007 study empirically
  2. Segment 2: Time (from Pt B to Pt C) [piping only] – calculated using plug flow
  3. Segment 3: Time through 1 MG [tank only] – calculated using 2011 tracer study-determined baffling factor
- Contact Time = (-0.028 Q + 75) [segment 1] + (2,094.03/Q) [segment 2] + (0.18 x V_{1MG}) / Q [segment 3]
  -where Q is sum of two individual effluent peak flows leaving 1 MG reservoir entering system
  -where V_{1MG} is the minimum volume determined in the 1 MG reservoir each day
- CT achieved = C x total contact time above; where C is chlorine residual measured in mg/l at 1st user (Point C)

Source Information

ID	Entry Points (Location where water enters distribution and is sampled)	Source Type	Availability (if seasonal, indicate begin/end dates)	Begin (M/D)		End (M/D)	
A	EP for WTP & Well	Surface	Permanent				
B	EP for Miller Road Wells	Ground	Permanent				

ID	Sources (Contributing to Entry Point)	Land Use*	Capacity (GPM)	Source Type	Availability
AA	South Fork Scappoose Creek	K		Surface	Permanent
AB	Gourley Creek	K		Surface	Permanent
AC	Lacey Creek	K		Surface	Permanent
AD	Dutch Canyon Well	G	275	Ground	Permanent
BA	Miller Road Test Well #1	I	256	Ground	Permanent
BB	Miller Road Production Well #2	I	200	Ground	Permanent
BC	Miller Road Well #3	I	180	Ground	Permanent

*Land Use Codes: (A) **Pristine Forest** (B) **Irrigated Crops** (C) **Non-Irrigated Crops** (D) **Pasture** (E) **Light Industry** (F) **Heavy Industry** (G) **Urban-Sewered Area** (H) **Rural On-Site Sewage Disposal** (I) **Urban On-Site Sewage Disposal** (J) **Rangeland** (K) **Managed Forest** (L) **Commercial** (M) **Recreational Use**

Yes No

☒ ☐ Has the water system implemented strategies (e.g., posting source area signs, notifying residents of Haz Waste collection events, provide residents information about maintaining their septic systems, abandoning unused wells, etc.) to protect their drinking water sources?

☒ ☐ Is the water system interested in protecting their drinking water sources from contamination? If yes, contact regional geologist at 541-726-2587.

Comments:

City hoping for increased and more effective communication from forestry owner in surface water intake watersheds, when applying chemicals that may affect City's water quality.
Conducted recent rehabilitation on Miller Road wells, resulting an 120 gpm increased total capacity for all 3 wells.
Currently working on a water right for Miller Rd well #3, has a limited license with Or Water Resources Dept now.

Well Information

Source ID#: SRC-	AD		BA		BB		BC					
Source Name:	Dutch Canyn Well		Miller #1		Miller #2		Miller #3					
Well log available?*	Yes		Yes		Yes		Yes		Choose an item.		Choose an item.	
Well log ID (e.g., COLU123, L12345)	L 48786		L 44949		L 37092		L 41159					
	Yes	**No**	**Yes**	**No**	**Yes**	**No**	**Yes**	**No**	**Yes**	**No**	**Yes**	**No**
Well active?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
Pitless adaptor?	☐	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐
● Sanitary seal & casing watertight?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
● Raw water sample tap?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
● Treated water sample tap? ☐N/A	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
● If vented, properly screened?	☐	☒	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
● Wellhead protected from flooding?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
Concrete slab around casing?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
Casing height ≥12-in. above slab/grade?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
Flowmeter?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
Pressure gauge?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
Pump to waste piping?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
● Well meets setbacks from hazards?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
If no, identify list of hazard(s) within the setback and the distance to the hazard……… HAZARD: DISTANCE (ft):												
Protective housing?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
If yes, does it have:												
Heat?	☒	☐	☒	☐	☒	☐	☐	☒	☐	☐	☐	☐
Light?	☒	☐	☒	☐	☒	☐	☐	☒	☐	☐	☐	☐
Floor drain?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
Well pump removal provision?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
Pump Type:	Verticle Turbine		Submersible		Submersible		Submersible		Choose an item.		Choose an item.	
Bearing lubrication:	Water		Water		Water		Water		Choose an item.		Choose an item.	
Pumping capacity (gpm):	275		200		200		200					

*If no well log available, record any known information regarding depth of well, depth of grout seal, year of installation, or casing diameter in the comments section below.

Comments:
Ensure Dutch Canyon wellhead has a properly screened down-turned vent – opening noted during survey, see photo on page 11.

Potential Sanitary Hazards

(From OAR 333-061-0050(2)(a)(E))

The following sanitary hazards are not allowed within 100 feet of a well or spring:

- Any existing or proposed pit privy
- Subsurface sewage disposal drain field
- Cesspool
- Solid Waste disposal site
- Pressure sewer line
- Buried fuel storage tank
- Animal yard, feedlot, or animal waste storage
- Untreated storm water or gray water disposal
- Chemical (including solvent, pesticides, and fertilizers)storage, usage, or application)
- Fuel transfer or storage
- Mineral resource extraction
- Vehicle or machinery maintenance or long term storage
- Junk / auto / scrap yard
- Cemetery
- Unapproved well
- Well that has not been properly abandoned or of unknown or suspect construction
- Source of pathogenic organisms
- Any other similar public health hazards

The following are not allowed within 50 feet of a well or spring:

- Gravity sewer line
- Septic Tank

Exemptions to these setbacks must be listed and documented within the plan approval letter and in an approved construction waiver standard.

If a surface water source is located within 500 feet of a well or spring, please note the water body name and the distance to the well or spring. All groundwater sources within 500 feet to a surface water source should be considered for potential surface water influence. Check the file for correspondence. If a review has been done indicate results in comment section. If not, contact the Springfield office 541-726-2587.

Dutch Canyon Wellhead Photo

Opening observed in vent in wellhead

Conventional & Direct Treatment Plant Inspection

☒ WTP inspection done with Water System Survey

☐ WTP inspection only

WTP ID:	A	WTP Name:	TP for WTP (Keys Rd WTP)
Date of inspection:	11/3/16	Inspected by:	James Nusrala
Total points given:	14	Plant operator:	Darryl Sykes

Points	Visit Frequency	Check One
Low range (0-15)	Every 3 years	☒
Mid range (16-25)	Annually	☐
High range (26 or more)	Every 6 months	☐

Comments:
Surface water plant last used on September 3rd, due to sediment removal work behind intake diversion dams and work on raw water transmission lines.

Source:

Describe Intake:	Three separate intakes with diversion dams and screens on each. Gourley and South Fork recently had sediment removed at intakes.
Describe pumping facilities:	All gravity flow to Keys plant, no pumping facilities.
Watershed control information: (protection plan, security measures, etc)	Locked gates on roads to all intakes. Intakes visited most days surface plant is on.
Factors affecting water quality: (algal blooms, logging, etc.)	Fall tannin, high raw water turbidity during winter rain storms.

Treatment:

☒ Coagulation — Chemical added: Alum

☒ Sedimentation basin ☒ Tube settlers ☐ Adsorption clarifier ☐ Solids contact clarifier

☐ pH Adjustment ☒ Flocculation ☒ Filter Media (☐single ☒dual/mixed ☐deep bed >60” anthracite)

☒ Corrosion control ☒ Other treatment — Describe: Caustic soda and fluoride

Peak instantaneous op. flow last year (gpm): 530 — Comments:

Filter Area (total) (ft^2): 397

Filter Loading Rate ($^{gpm}/_{ft2}$): 1.34

Log removal credit given *Giardia*: 2.0 *Crypto*.: 2.5

What was the peak instantaneous operating flowrate at time of treatment plant evaluation (gpm): 700

Based on: ☒ CPE ☐ Plan review ☐ WTP evaluation/rating form Date: 3/30/93

Comments:
The Keys Road plant is a Keystone conventional filtration plant built in 1979. The plant has been rated as a 2.0-log *giardia* reduction direct filtration plant through the Comprehensive Performance Evaluation performed by DWS, with the sedimentation phase limiting (504 gpm theoretical flow from CPE, < 80 % of peak instantaneous operating flow in 1993 of 700 gpm-Type 3 unit process). Filter design – 750 gpm. Filter area measured in 2013 WSS

Conventional/Direct Treatment Plant Continued: **WTP-** A

Yes	No		If no, check points
☒	☐	Is raw water turbidity data collected at least daily? ☒ On-line ☐ Bench-top Raw water turbidity measured daily when surface plant running with calibrated bench top. Summer - < 1.0 NTU, winter – up to 6 NTU (maximum to treat).	☐ **3 pts**
☐	☐	● For 2.5-log plants only: Is settled water turbidity measured at least daily? ☒ N/A	☐ **5 pts**
☐	☐	When average annual raw water turbidity is ≤ 10 NTU, is settled water turbidity ≤ 1.0 NTU?	☐ **2 pts**
☐	☐	When average annual raw water turbidity is > 10 NTU, is settled water turbidity ≤ 2.0 NTU?	☐ **2 pts**
☒	☐	● Are turbidity compliance standards met? (<0.3 NTU 95% of time; all < 1 NTU)	☐ **10 pts**
☐	☒	Are filter Optimization goals met? (≤ 0.10 NTU 95% of time; always ≤ 0.30 NTU) ☐ CFE ☒ IFE	☒ **4 pts**
☒	☐	● Is CFE monitoring location acceptable (prior to any storage)? Max 0.27 & 95th %tile 0.18 NTU (Aug 1 '15 – July 31 '16); reports highest of each IFE every 4 hours (and max of day too), because CFE is after merging w/ Dutch Canyon well. Uses slider bar on SCADA to determine highest of each 4-hour IFE reads and highest IFE of the day.	☐ **5 pts**
☒	☐	Is each IFE turbidity always below triggers? If no, check box below: ☐ Turbidity > 1.0 NTU in 2 consecutive 15-min readings ☐ > 10,000 population only: Turbidity > 0.5 NTU in 2 consecutive readings 1st 4 hrs after startup ☐ Turbidity > 1.0 NTU in 2 consecutive 15-min readings for 3 months in a row ☐ Turbidity > 2.0 NTU in 2 consecutive 15-min readings for 2 months in a row	
☒	☐	Can chart recorder document turbidity > 1.5 NTU? ☐ N/A Hach IFE controllers can record > 1.5 NTU, but SCADA tops out at 1.0 NTU for IFEs. Can use an SD card in SC200 controllers on IFEs to download NTU.	
☒	☐	Are chemical dosages adjusted with water quality changes (jar test or equivalent)? Process identified: Uses streaming current meter, with a minus 20 to minus 40 as the sweet spot. Can adjust alum dose accordingly to target ideal SCM setting. For example, if at minus 70, will increase alum speed setting on feed pump to increase dose. Alum feed is flow – paced to account for varying plant flow to maintain consistent dose.	☐ **3 pts**
☒	☐	If using alum, is raw water alkalinity collected at least weekly? ☐ N/A Normally 20-30 mg/L as $CaCO_3$, data kept on site.	☐ **3 pts**
☐	☒	Does the operator know all chemical dosages applied in mg/L? Current chemical dose calculations using conservation of mass $(C \times Q)_{plant} = (C \times Q)_{feed}$ may not be accurate, as using chemical feed pump rate and not calibrating feed pumps regularly.	☒ **3 pts**
☐	☒	Are feed pumps calibrated at least annually?	☒ **3 pts**
		How is backwash initiated?-normally on time, uses headloss & NTU as secondary triggers (0.7 NTU end of run) ☐ Turbidity level: ☒ Headloss: ☒ Time: 48 hr	
☒	☐	Is total plant flow adjusted when filters are taken off-line for backwashing? – plant not backwashing off-line	
☒	☐	Is evidence of air binding absent during backwash? – mud balls and sediment observed in filter sides.	
☒	☐	Does the plant have filter to waste piping?	☐ **3 pts**
☒	☐	If yes, is the duration of filter-to-waste cycle based on turbidity profile results? ☐ N/A What is the criteria for putting filters back on-line? Tries to reach ≤ 0.09 NTU on 1720E controller most of the time to end FTW.	☐ **3 pts**

Yes	No	**Conventional/Direct Treatment Plant Continued:** WTP- A	If no, check points
☒	☐	• Are filter profiles conducted after backwash at least quarterly?	☐ **5 pts**
☐	☒	Are optimization goals immediately after backwash met? If no, check goal NOT met: For all conventional/direct plants: ☐ Max spike ≤ 0.30 NTU ☒ ≤ 0.10 NTU within 15 minutes For plants with filter-to-waste capability: ☐ Return to service ≤ 0.10 NTU August 2016 filter #2 did not return to ≤ 0.10 NTU within 15 minutes.	☒ **4 pts**
☐	☐	• If recycling filter backwash water, is return location prior to chemical addition? ☒ N/A Backwash & filter to waste to settling pond and then to City wastewater plant.	☐ **5 pts**
☒	☐	• Are turbidimeters calibrated according to factory specifications or at least quarterly?	☐ **5 pts**
☒	☐	Are calibration standards valid (not expired)?	
☒	☐	Is flow through turbidimeter within manufacturer's range? ☐ N/A (bench top or portable meter) Recent quarterly calibrations stored on NTU meter controllers, between 300 and 400 mL/min noted in recent flow through measurements.	
☒	☐	• Are CT's calculated correctly?	☐ **10 pts**
☒	☐	• Is contact time based on tracer study or adequate alternative?	
☒	☐	• pH, temperature, and chlorine residual measured at or before 1st user?	
☒	☐	• Is there a flow meter on effluent side of clearwell or adequate alternative (describe)?	
☒	☐	Is corrosion control practiced?	
☒	☐	• Is it operated within parameters set by DWP? ☐ N/A Describe method of corrosion control used: System meets minimum pH of 7.2 at both entry points. Needs to take 3 distribution pH and alkalinity w/ lead and copper tap samples in future, has to also meet 7.2 min. pH in distribution	☐ **5 pts**
☒	☐	• Do all under-certified operators follow a written decision-making protocol as established by DRC? ☐ N/A (all operators are certified at the level required for the plant) Written protocols for under-certified operators at treatment plant completed. Needs to complete written protocols for distribution system operators.	☐ **5 pts**
☒	☐	• Are standard plant operating procedures written and followed?	☐ **5 pts**
☐	☒	Are operators on site during all hours of plant operation?	
☒	☐	• If no, is there an alarm for low chlorine and high turbidity? (> 3300 pop. for chlorine) ☒ Low chlorine ☒ High turbidity ☒ Plant shutdown ☒ Auto-dial 0.15 IFE hi alarm calls operators, 1.0 NTU IFE automatically shuts down plant; 0.2 mg/L low and 1.5 mg/L high first user chlorine alarm set points.	☐ **5 pts**
		Total Points =	**14**
☒	☐	AWOP fact sheet provided to operator?	

Comments:

Disinfection

No #	Disinfection Method*	Location	Disinfection Source Water	Residual Maintenance	Other Purpose	Proportional to Flow	Dosage Recorded
1	Sodium Hypochlorite	Keys Rd WTP – Dutch Canyon Well	☐	☒	F423	☒	☒
2	Sodium Hypochlorite	Keys Rd WTP – surface water	☒	☐		☒	☒
3	On-site Generated Sodium	Miller Rd WTP - pre	☐	☒	F 423	☒	☒
4	Sodium Hypochlorite	Miller Rd WTP – post-not used	☐	☒		☒	☒

Yes	No	**Chlorine residuals** ☐ N/A
☒	☐	• Is a DPD or other EPA approved method used?
☒	☐	• NSF 60/61 certified (or equivalent)?
☒	☐	• Are entry point residuals recorded at least once per day (SWTR, GWR 4-log)? ☐N/A
☒	☐	• Is entry point residual monitoring continuous if population > 3,300? ☐N/A
☒	☐	• Are distribution residuals recorded at least twice weekly?
☒	☐	Are on-line chlorine analyzers verified weekly with DPD type or EPA approved test kit? ☐N/A

Yes	No	**Chlorine gas** ☒ N/A	Yes	No	
☐	☐	Separate room for gas storage and feeder?	☐	☐	Gas cylinders properly secured?
☐	☐	Fan with on/off switch outside?	☐	☐	Door that opens out?
☐	☐	Vent located next to the floor?	☐	☐	Self-contained breathing apparatus?
☐	☐	Door with a window?	☐	☐	Air scrubber system?

Yes	No	**UV** ☒ N/A
☐	☐	• Does all water contact UV (no bypass)?
☐	☐	• Is lamp sleeve cleaned?
☐	☐	• Is lamp replaced per manufacturer?
☐	☐	• Intensity sensor with alarm or shut-off?

Yes	No	**CT Evaluation for disinfection** ☐ N/A
☒	☐	• Is contact time based on a tracer study or adequate alternative? ☐ N/A
		Describe adequate alternative method for contact time: NA
☒	☐	• Is there a flow meter on effluent side of clearwell /contact chamber or adequate alternative?
		Describe adequate alternative method for flow rate: NA
		Tracer study demand flow (gpm): 1200 for 2MG / 1250 for 1 MG
☒	☐	Have tracer study parameters changed? – see below
☒	☐	• (SW only) Are pH, temp, and chlorine residual measured daily before or at the first user? WTP
☒	☐	• Are CT values being calculated correctly? – rounds correctly for CT
☒	☐	• Are CT values met at all times (SWTR, GWR 4-log)?

Comments:

Keys Rd WTP: Peak hourly effluent flow from 1 MG tank determined by sliding over SCADA flow for highest 60-min. average of the day – 1,133 gpm in June 2016, minimum level in 1 MG was 16.7 feet or 774,880 gal; Tracer study volumes: 2 MG (1.5 MG +pipe), 1 MG (626,400 gal + pipe)
Contact time formula (Pipe&2 MG + comb. yard pipe + 1 MG): [(-0.028 x Q) + 75] + [2,094.03 / Q] + [(0.18 x Vmin in 1 MG) / Q];
where Q is peak demand flow-sum of 2 meters after 1 MG tank, 2007 T. Study determined Pipe & 2MG formula, 2011 T. Study determined 1MG formula, middle formula is for 356.6 ft of 12-inch yard piping segment, which is from 2007 T. Study. Does not bypass either tank. Dutch Canyon well receives 1-log *giardia* and 4-log viral CT through Keys Rd 2 tanks.
Miller Rd WTP: Minimal contact time between Miller Rd wellfield WTP and 1st user, not required, wells coliform absent.

Treatment

Code / Purpose / Process Used*	Chemical Added**	Location in System
P240 | Particulate Removal (SWTR) | Coagulation	Alum	Keys Rd WTP
P360 | Particulate Removal (SWTR) | Flocculation	N/A	Keys Rd WTP
P660 | Particulate Removal (SWTR) | Sedimentation	N/A	Keys Rd WTP
P345 | Particulate Removal (SWTR) | Filtration, Rapid Sand	Nalclear 8170PULV	Keys Rd WTP
F423 | Iron Removal | Hypochlorination, Pre	Sodium hypochlorite	Keys Rd WTP
F560 | Iron Removal | Permanganate	Pot. Permangenate	Keys Rd WTP
F343 | Iron Removal | Filtration, Greensand	NA	Keys Rd WTP
C503 | Corrosion Control | pH/Alkalinity Adjustment-Caustic Soda	Caustic Soda	Keys Rd WTP
D401 | Disinfection for Surface Water/GWUDI | Gaseous Chlorination, Post	Sodium hypochlorite	Keys Rd WTP
Z380 | Other | Fluoridation	Fluoride	Keys Rd WTP
F423 | Iron Removal | Hypochlorination, Pre	Sodium hypochlorite	Miller Rd WTP
F560 | Iron Removal | Permanganate	Pot. Permangenate	Miller Rd WTP
F343 | Iron Removal | Filtration, Greensand	NA	Miller Rd WTP
C502 | Corrosion Control | pH/Alkalinity Adjustment-Soda Ash	Soda Ash	Miller Rd WTP
Z380 | Other | Fluoridation	Fluoride	Miller Rd WTP

*See “Treatment Plant Inspection” page for details on filtration. **See “Disinfection” page for details on disinfection equipment.

Yes	No		
☒	☐	Has treatment changed?	Filter aid changed to Naclear 8170 PULV
☐	☒	Is lab equipment for on-site analysis appropriate?	See below
☒	☐	Is equipment maintained properly?	______
☒	☐	Is redundant equipment available?	______
☒	☐	● Are chemicals NSF Standard 60 certified or equivalent? (☐N/A - no chemicals are used)	
☒	☐	Does system practice corrosion control?	
☒	☐	● Is corrosion control operated within parameters set by DWS? ☐N/A	

Describe method of corrosion control (if applicable) – Keys – caustic soda; Miller – soda ash

Had two excursions below 7.2 minimum at Miller Rd in Sept. 2014 due to plug in soda ash feed line, quickly fixed, Needs to take 3 distribution pH and alkalinity samples with LCR tap samples in summer 2017; min. 7.2 pH in distribution also. Will also need to begin collecting entry point samples for alkalinity at Miller Rd at least once every two weeks as alkalinity is adjusted with soda ash addition. DWS will set alkalinity minimum at Miller after reviewing lead and copper tap samples.
Hach DR 800/DR 2010 used for chlorine residuals, fluoride, and iron readings. Hach DR 800 – Miller and Hach DR 2010 – Keys – use SPADNS method - EPA compliant standard method for fluoride, Orion star A211, Hach benchtop turbidimeter, alkalinity test kit, and jar test mixer at Keys.

Records Kept:

Yes	No		Yes	No	
☒	☐	Dosages	☒	☐	Flowrate
☒	☐	Raw pH	☒	☐	Treated pH
☒	☐	Raw temperature	☒	☐	Treated temperature
☒	☐	Raw turbidity and/or particle counts	☒	☐	Treated turbidity

Comments:

Sodium hypochlorite at Miller is on-site generated
Chemical manufacturers:
-Northstar: alum, sodium hypochlorite, & caustic soda; Nalco – Naclear 8170 PULV – Keys
-Shanghai Mintchem – sodium fluorid, Organic Industries – potassium permanganate – Keys & Miller
-Solvay – soda ash, Morton – salt for Cl2 - Miller

Storage and Pressure Tanks

Number	Name	Tank Type*	Tank Material	Year Built	Volume (gal.)
1	Keys Road WTP – 1.0 MGal	(G) Ground	Concrete	1968	1.0 MG
2	Keys Road WTP – 2.0 MGal	(G) Ground	Concrete	2004	2.0 MG
3	High Zone Reservoir – Small	(G) Ground	Steel	1968	0.3 MG
4	High Zone – Big	(G) Ground	Steel	2004	0.35MG
				Total Volume:	3.65 M Gal

Reservoir Number:	1		2		3		4			
Reservoir Features	**Yes**	**No**	**Yes**	**No**	**Yes**	**No**	**Yes**	**No**	**Yes**	**No**
Fence/gate?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐
● Hatch secured (e.g. locked, bolted, etc)?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐
● All tank access points watertight?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐
● Screened vent?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐
Overflow?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐
● Overflow protected (screen/flap/valve)?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐
Drain to daylight?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐
Water level gauge?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐
Bypass piping?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐
Alarm for high or low levels?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐
Separate inlet/outlet?	☒	☐	☒	☐	☐	☒	☒	☐	☐	☐
Approved interior coating?	N/A		NA		☒	☐	☒	☐	☐	☐
Exterior in good condition?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐
Annual interior/exterior inspection?	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐
Cleaning schedule?	☐	☒	☐	☒	☐	☒	☐	☒	☐	☐
Continuously disinfected? (● post '81 redwood)	☒	☐	☒	☐	☒	☐	☒	☐	☐	☐
Pressure Tanks	NA									
Accessible for maintenance?	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
Bypass piping?	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
Drain?	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
Pressure relief device?	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
Air bladder/diaphragm?	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
Valve for adding air?	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐

Comments

Fall 2016 photos document locked and watertight hatches and adequate screening on rooftop vents at all storage reservoirs.

Distribution System Information

Service Area and Facility Map

Yes	No	
☒	☐	Does the system have a service area and facility map (indicate features on map):

- ☒ Water lines (including size and material)
- ☒ Treatment facilities
- ☒ Storage facilities (reservoirs)
- ☒ Sampling points
- ☒ Sources-wells & withdrawal points
- ☒ Pressure zones
- ☒ Pressure regulating valves
- ☒ Booster pumps

Distribution Data

Yes	No		Comments
☒	☐	● System pressure ≥ 20 psi?	60-120 psi, varies by elevation
☐	☒	Water system leakage <10%?	15 %
☒	☐	Hydrants or blowoffs on all dead ends? ☐ N/A	blowoffs
☒	☐	Routine flushing? (How often)	Once / 2 yrs, or more frequently by complaints
☒	☐	Adequate valving?	
☒	☐	Routine valve turning? (How often)	With flushing of lines
☐	☒	Does the distribution system have asbestos cement (AC) pipe?	Ductile iron, PVC, and tar wrapped steel

If yes, *verify asbestos sampling is completed on Water Quality Monitoring Page (CWS, NTNC).*

Cross Connection Control (CWS, NTNC, and TNC)

Yes	No	N/A		Comments
☒	☐	☐	● Devices tested annually? (CWS, NTNC, TNC)	
☒	☐	☐	● Ordinance or enabling authority? (CWS)	
☒	☐	☐	● Annual Summary Report submitted? (CWS)	2013 – 2015 received by DWS
☒	☐	☐	● Certified Cross Connection Control Specialist? (CWS $\geq$ 300 connections)	Doug Nassimbene

Comments:

Customers test residential assemblies using City-provided contracted testers, Crow Water, Inc. tests City assemblies.
City sends out reminders and notices to discontinue water to ensure annual testing of all assemblies.
City tracks testing results on a spreadsheet.
2015 ASR: Has more Reduced Pressure assemblies than 25 high hazards – good!.
Booster pump control valve at Dutch Canyon well and GA solenoid controlled throttling valve at Miller Road Well #2: Finished (treated water) not in contact with raw well water. Therefore, no high hazard according to Table 42 in DWS rules requiring reduced pressure or air gap assembly required. Existing double check cross connection control assemblies at both sites adequate.
Reduced pressure cross connection control assemblies used at both Keys and Miller Road treatment plants with finished treated used as carrier water to mix with chemicals.

Water Quality Monitoring

Entry Point Sampling:	N/A	EP-A for Keys WTP & Well: *Frequency*	EP-A: *Next Test Due*	EP-B for Miller Rd Wells: *Frequency*	EP-B: *Next Test Due*
Nitrate	☐	One annually	2017, 2018. . .	Once annually	2016, 2017. . .
Arsenic	☐	Once every 9 yrs	2019	Once / 9 yrs	2017
Inorganic Chemicals (Including Nitrite) (sw)	☐	Once every 9 yrs	2019	N/A	
Inorganic Chemicals (Including Nitrite) (gw)	☐	N/A		Once / 9 yrs	2016
SOCs	☐	2 cons qtrs. / 3 yrs	2019	2 cons qtrs /3 yrs	2019
VOCs (sw)	☐	One annually	2017, 2018. . .	NA	
VOCs (gw)	☐	NA		Once / 3 yrs	2016
Radionuclides (Community Water Systems Only):					
Gross Alpha	☐	Once every 9 yrs	2016	Once / 6 yrs	2019
Radium 226/228	☐	Once every 9 yrs	2016	Once / 9 yrs	2022
Uranium	☐	Once every 9 yrs	2016	Once / 9 yrs	2022
Distribution System Sampling:		*Frequency*		*Next Test Due*	
Coliform Bacteria	☐	8 a month		ongoing	
Asbestos (for AC pipe/asbestos geologic areas)	☒				
TTHMs and HAA5s	☐	2 annually: March-Branch/Dec-Skyway		Dec '16-Skyway; Mar '17 - Branch	
Lead and Copper, # sites: 20	☐	Every 3 years in summer		June 1 – Sept 30, 2017	
Other Sampling:		*Frequency*		*Next Test Due*	
TOC- Raw water Keys WTP	☐	Once a quarter		4th qtr., 2016. .	
Turbidity – combined NTU – Keys WTP-surface	☐	Once every 4 hours		ongoing	
Source Water Coliform – Dutch Cnyn & all Miller wells	☐	Once annually		2016, 2017. . .	
Other (specify) Raw alkalinity – Keys plant	☐	Once weekly		Ongoing (kept onsite)	

Yes	No	
☒	☐	• Is all required monitoring current?
☒	☐	Are samples collected at the correct locations in the system?
		Discuss correct sampling locations for all sampling (SRC, EP, DIST)
		Discuss proper way to collect representative samples at all locations
		Discuss possible sample reductions

Yes	No	
☐	☐	• Have all MCL violations or LCR AL exceedances been addressed? ☒ N/A No MCL's or ALE's
☒	☐	DBP's collected at correct locations? ☐ N/A
☒	☐	• Does the system have a written coliform sampling plan?

Does the plan include:

Yes	No		Yes	No	
☒	☐	Brief narrative	☒	☐	Rotation schedule
☒	☐	Distribution map	☒	☐	Repeat locations
☒	☐	Sample site locations	☒	☐	Source locations ☐ N/A

Comments:
Remind to pull water quality parameters (pH and alkalinity) at 3 taps with LCR tap sampling in future (can use coliform sites); Sample for alkalinity at entry point at Miller Rd due to soda ash addition. Continue to notify individual residents of lead results 30 days of receiving them and certify w/in 90 days to DWS, we did get last sample cert.
Fluoride max at 1.3 ppm (>0.7 ppm) in 2015 CCR, and Miller fluoride results are on same days as corrosion control when Miller plant on.
Fluoride: Split samples off by 30 % with DEQ sample, 2016: DEQ-0.22, City-0.32 mg/L
Lead/copper: sampling same or similar sites from 2011 to 2014, good!

Management & Operations

O&M Manual and Emergency Response Plan

Yes	No	
☒	☐	• Does system have an operation and maintenance manual? – 2005 &2015 – Miller; 2015 Keys
☒	☐	• Does system have an emergency response plan?
☒	☐	Do any system components have auxiliary power?
		If yes, describe: At both Miller and Keys plants, Dutch Canyon well and booster pumps (portable)

Operator Certification

Yes	No	N/A	
☒	☐		• Is the DRC identified and certified at the appropriate level?
		☒	If the DRC is a contract operator, how do they work with the system?
☐	☒	☐	• Does system have written protocols for under-certified operators? – for treatment, not for dist.

Plan Review/Master Plan

Yes	No	N/A	
☒	☐	☐	• Have all major modifications been approved by DWS?
☒	☐	☐	• Does the system have a current (<20 yr. old) master plan? (Not required if < 300 connections)
			What year was the plan completed? 1997

Compliance Status

Yes	No	N/A	
☒	☐		• Is water system in compliance (all orders resolved and not a priority non-complier)?
☒	☐	☐	• Does the system issue public notice as required?
☒	☐	☐	• Are consumer confidence reports sent to users each year?

Comments:

Make sure DWS receives annual CCR by July 1st of following year, same as date all users receive it.
Master plan due in 2017, last in 1997.
Has current waterline plan review exemption.

Appendix B

WATER LOSS CONTROL PLAN

7/1/2019

2 Year Water Loss Reduction Plan

History

Within recent years, the City of Scappoose has undergone many changes. Those changes include: new City Management and Staff, several new housing developments and substantial new commercial development. Subsequently, the City has been working to update its Master Plans for Water, Wastewater and Stormwater.

During these updates, City Staff and Engineers from Carollo have identified a 10% or greater increase in water loss from previous years. Since 2015, the City has been experiencing between 33% - 38% loss.

City Staff believe that these losses are a combination of Real Losses, Apparent Losses and Non-Revenue Authorized Consumption.

- **Real Losses**: much of the City's distribution system is aged and has zones with pressures exceeding 100 psi. City crews are responding to approximately 20 leaks per year.
- **Apparent Losses**: meter inaccuracy, water theft, recording and computing errors.
- **Non-Revenue Authorized**: lack of usage recording for system flushing, fire fighting, fire training and construction of new infrastructure.

The City of Scappoose operates from a Budget that renews annually on July 1st. Therefore, this Plan will follow projects identified the City's fiscal year Budgets.

City of Scappoose 33568 E Columbia Avenue Scappoose Oregon 97056 503-543-7146 Fax 503-543-7182

Plan

2019-20

- Develop a Leak Detection Program
- Contract a City-wide leak detection update
- Continue to replace/install new remote read meters
- Evaluate the City's current data logging and billing practices
- Coordinate with the Scappoose Fire Department to develop and method of tracking water usage for fires and training
- Implement better methods of tracking water used for Construction of both City and Private projects
- Identify pipelines in need of replacement
- Upgrade Water Treatment Plant SCADA systems to improve metering accuracy of water production and potentially reduce backwash cycles and associated non-revenue authorized water use

2020-21

- Evaluate 2019-20 progress
- Continue leak detection program
- Continue water meter replacement
- Begin engineering of pipeline replacement and pressure zone improvement projects
- Begin construction of pipeline replacements

****Note – This Plan to be updated annually***

Appendix C

CITY ADOPTING RESOLUTION AND ORDINANCE

Appendix D
AGENCY COMMENTS AND RESPONSES

PUBLIC HEALTH DIVISION
Drinking Water Services
Kate Brown, Governor

Oregon Health Authority

800 NE Oregon Street, #640
Portland, OR 97232-2162
Phone: 971-673-0462
Fax: 971-673-0694
healthoregon.org/DWP

10 January 2020

Alexandra Rains
City of Scappoose
33568 East Columbia Avenue
Scappoose Oregon 97056

Re: **2019 Water Master Plan (PR# 168-2019)**
City of Scappoose (WS ID# 00792)
Concurrence with Master Plan

Dear Ms. Rains:

Thank you for your submittal to the Oregon Health Authority's Drinking Water Services (DWS) of plan review information for the Water System Master Plan for the City of Scappoose. On 5 November 2019, our office received a copy of the 2019 Master Plan. A plan review fee of $4,125 was also received.

The Master Plan must consider at least a 20-year planning horizon out to the year 2039. The plan includes a system description, future demand estimates, seismic analyses, and CIP project lists with cost estimates. Upon review of the Master Plan, it appears the criteria listed in Oregon Administrative Rules (OAR) 333-061-0060(5) have been met.

If you have any questions, please feel free to call me at 971.673.0462.

Sincerely,

Pete Farrelly, PE
Regional Engineer
Drinking Water Services