FINAL REPORT | MAY 2023

City of Rainier
Water Master Plan Update

PREPARED FOR

City of Rainier

PREPARED BY

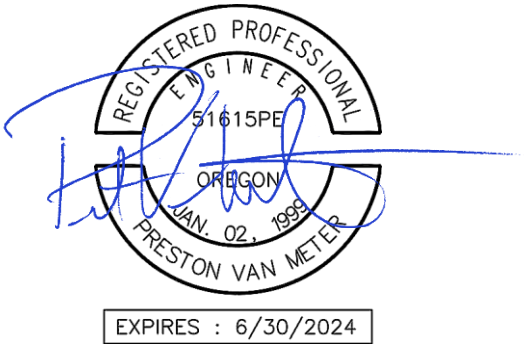
WEST YOST
Water. Engineered.

City of Rainier
Water Master Plan Update

Prepared for

City of Rainier

Project No. 962-60-20-01



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Appendix B. Seismic Evaluation of Existing Structures Technical Memorandum, May 2, 2023 Ace Engineering LLC

Appendix C. Columbia River Raw Water Intake Facility – Priliminary Cost Estimate and Proposed New Intake Facility Layout

LIST OF ACRONYMS AND ABBREVIATIONS

AC	Asbestos Cement
ADD	Average Day Demand
AF	Acre-Feet
ALA	American Lifelines Alliance
AWWA	American Water Works Association
CCI	Construction Cost Index
Cfs	Cubic Feet Per Second
CI	Cast Iron
CIP	Capital Improvement Program
City	City of Rainier
COP	Copper
County	Columbia County
CSZ	Cascadia Subduction Zone
DI	Ductile Iron
DIP	Ductile Iron Pipe
DWS	Drinking Water Services
ELA	Engineering, Legal, and Administrative Services
EPA	Environmental Protection Agency
ENR	Engineering News Record
ERDIP	Earthquake Resistant Ductile Iron Pipe
fps	Feet Per Second
GALV	Galvanized
GIS	Geographic Information System
gpd	Gallon Per Day
HDPE	High-Density Polyethylene
MDD	Maximum Day Demand
MG	Million Gallons
mgd	Million Gallons Per Day
msl	Feet Mean Sea Level
NRW	Non-Revenue Water

OFC	Oregon Fire Code
OHA	Oregon Health Authority
ORP	Oregon Resilience Plan
PHD	Peak Hour Demand
POD	Point of Distribution
PRS	Pressure Regulating Stations
PRV	Pressure Regulating Valves
psi	Pounds Per Square Inch
PVC	Polyvinyl Chloride
RR	Repair Rate
SR	State Route
STL	Steel
TDH	Total Dynamic Head
UNK	Unknown
UPC	Uniform Plumbing Code
USG	United States Gypsum
WA	Washington
WMP	Water Master Plan
WTP	Water Treatment Plant

CHAPTER 1
Introduction

1.1 WATER SYSTEM MASTER PLAN UPDATE PURPOSE

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

Evaluations and recommendations presented in this WMP Update are based on information collected in late-2020, including historical data and records, record drawings, past surveys and reports, current Geographic Information System (GIS), and results from requested field inspections/data collection collected for this WMP Update. Any updates or improvements completed within the City's water system and/or operational changes implanted in 2021 have not been incorporated as part of this WMP.

1.2 WATER SYSTEM MASTER PLAN OBJECTIVES

The objectives of this WMP are to:

- Evaluate historical water meter data to develop current and estimated future water system average and peak demands;
- Identify operational and performance criteria to guide water system evaluations;
- Develop a GIS-based water system hydraulic model to complete evaluations;
- Analyze the existing distribution system to evaluate the City's ability to meet current and future demands;
- Identify system deficiencies and recommend upgrades to meet operational and performance criteria; and
- Develop a comprehensive CIP to address existing system deficiencies.

1.3 AUTHORIZATION

West Yost was authorized to prepare this WMP by the City on November 9, 2020.

1.4 REPORT ORGANIZATION

This WMP is organized into the following chapters:

- Chapter 1: Introduction
- Chapter 2: Existing Water System
- Chapter 3: Water System Performance Criteria
- Chapter 4: Water System Analysis
- Chapter 5: Seismic Risk and Mitigation Plan
- Chapter 6: Water System Capital Improvement Program

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

- Appendix A: Seismic Hazards Evaluation

1.5 ACKNOWLEDGEMENTS

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

- Sue Lawrence, Public Works Director, City of Rainier
- John Dewey, Water Operator, City of Rainier

CHAPTER 2
Existing Water System

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

- Existing Service Area
- Existing Water Supplies
- Existing Water Demands
- Existing Water System Facilities
- Organization Overview

2.1 EXISTING SERVICE AREA

The City is located within Columbia County (County), Oregon, about 40 miles north of Portland and 40 miles east of Astoria. The City is situated on the south bank of the Columbia River, across from Longview, Washington, with forested hills to the east, south, and west. State Route (SR) 30, the Columbia River Highway, runs in an east-west direction through the City along the Columbia River. Washington (WA) SR 433 bridges the Columbia River, connecting the City to Longview via the Lewis and Clark Bridge.

The City's existing water service area is approximately 5.13 square miles and is shown on Figure 2-1. The existing water service area is generally contiguous with City limits. Elevations within the City limits range from approximately 1,100 feet mean sea level (msl) in the hills south of the Fox Creek Reservoir to approximately 10 feet msl along the river. However, a majority of the City's water service is provided to customers at elevations between 20 feet msl and 400 feet msl.[1]

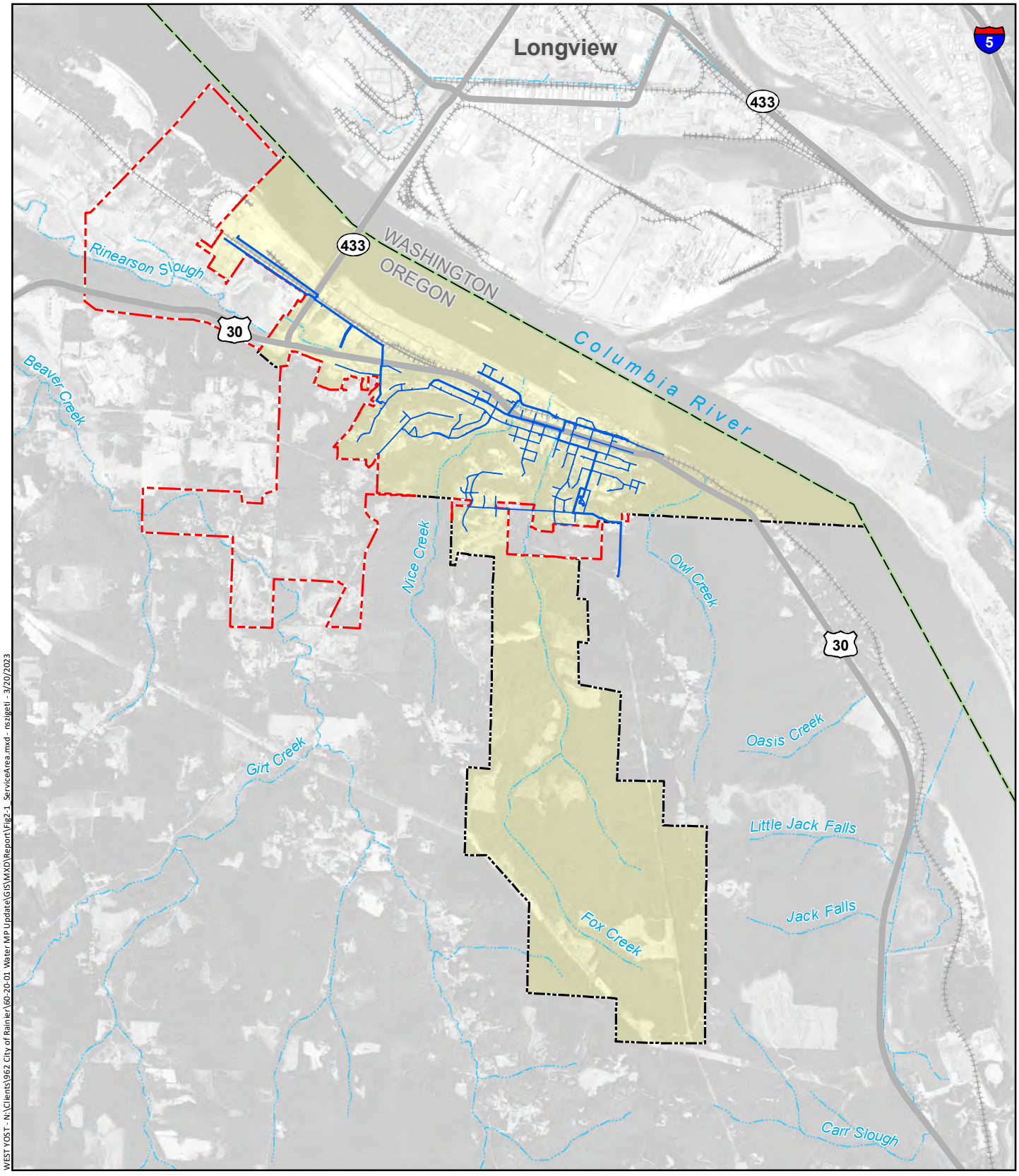
2.2 EXISTING WATER SUPPLIES

The City's existing water supply portfolio includes surface water from the Columbia River and Fox Creek. The following sections briefly describe these water sources and the City's drinking water quality and compliance history.

2.2.1 Sources of Water Supply

The City holds existing water rights to surface water from the Columbia River and Fox Creek, each of which are described in the following sections. Fox Creek was the City's original source of water, but the Columbia River has been the City's primary source of water since 2009.

[1] Source: USGS National Elevation Dataset - 2009 OR Northern Coast, downloaded on 12/11/2020.



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Symbology

- Pipelines
- City Limit
- Urban Growth Boundary (UGB)

N

0 2,000 4,000

Scale in Feet

WEST YOST

Figure 2-1

Existing Water Service Area

City of Rainier
Water Master Plan Update

2.2.1.1 Fox Creek

The City holds two existing water rights for municipal use from Fox Creek that total 1.25 cubic feet per second (cfs), or approximately 0.81 million gallons per day (mgd).[2] During the dry season, the Fox Creek watershed is typically unable to produce enough water to satisfy the City's water right. Therefore, the City also holds a storage permit for Fox Creek of 14.5 acre-feet (AF),[3] or 4.7 million gallons (MG).

A timber impoundment on Fox Creek was constructed in 1910 to create the Fox Creek Reservoir, which stores the maximum raw water storage of 14.5 AF, as allowed per the City's existing storage permit. From the reservoir, raw water can be conveyed through a 6-inch pipeline, constructed from cast iron (CI) and high density polyurethane (HDPE) pipeline, to treatment at the City's water treatment plant (WTP), where it would historically be blended with water from the Columbia River.

Water from Fox Creek has not been used by the City since 2009. A new WTP was constructed in 2000, which removed the raw water premixing reservoir and the ability to blend water supply sources. The differing water qualities between the two City's two water supply sources, as described in Section 2.2.2, has historically required blending for effective treatment of Fox Creek water, and the City has elected to rely solely on its Columbia River water supply since 2009.

2.2.1.2 Columbia River

The City holds an existing water right for municipal use from the Columbia River for 3.10 cfs, or approximately 2.0 mgd.[4]

The City diverts water from the Columbia River for treatment at the City's WTP. Raw water is pumped from a wet well intake structure and conveyed through a 10-inch ductile iron pipeline to the WTP. Since 2009, the City has depended solely upon the Columbia River to meet the City's water demand.

2.2.2 Drinking Water Quality and Compliance History

The City's raw water sources, Fox Creek and the Columbia River, require full treatment for use as a municipal water supply per state and federal regulations. Historically, the Fox Creek supply was a higher quality surface water source than the Columbia River, as the small watershed was well protected and undeveloped. Columbia River water has historically contained more pollutants from industrial use and urban development upstream of the City.

The Fox Creek Reservoir is a small impoundment with differing chemistry (pH, alkalinity, hardness, and turbidity) than the Columbia River, and the small reservoir experiences temperature fluctuations over the course of each day. In 2009, the City halted use of Fox Creek surface water over water treatment concerns to rely solely upon the Columbia River. Historically, the City blended the Fox Creek and Columbia River water supply sources to effectively treat the raw water. Since the WTP improvements in 2000, the WTP does not have raw water mixing capabilities. In addition, the chemical feed system for pH control has not

[2] The City holds water rights to Fox Creek through the Oregon Water Resources Department, Permit Numbers 21029 (for 0.30 cfs) and 34978 (for 0.95 cfs), with priority dates of 1/3/1952 and 8/10/1970, respectively.

[3] The City holds a storage permit through the Oregon Water Resources Department, Permit Number R-1319.

[4] The City holds water rights to the Columbia River through the Oregon Water Resources Department, Permit Number 33386 (for 3.10 cfs), with a priority date of 3/22/1968.

been needed to treat the Columbia River water and is currently disabled at the WTP. An upgrade of the pH control chemical feed system will be required to put it in use as the system is no longer serviceable. Therefore, differences in the water quality (i.e., pH) and temperature between the supply sources could interfere with operations at the City's WTP if Fox Creek is again used as a drinking water source for the City. It is presently unknown if the existing WTP would be able to properly treat Fox Creek water to acceptable state and federal water quality standards. It is recommended that the City conduct a study of the Fox Creek water supply to: (1) evaluate the existing water quality of Fox Creek; (2) determine improvements to the WTP required to treat Fox Creek supply; and (3) establish an operational plan for the WTP to maintain acceptable water quality within the City's distribution system.

2.3 EXISTING WATER DEMANDS

In recent years, the City has used treated Columbia River supply to serve customer water demands. Water production consists of the total treated surface water, whereas water consumption is equal to the total metered water use. The difference between production and consumption is non-revenue water (NRW). Additionally, customer water demands fluctuate daily and seasonally and are described by demand peaking factors.

The following subsections detail the City's historical production, consumption, and NRW, in addition to historical and recommended peaking factors.

2.3.1 Water Production

Table 2-1 summarizes the City's estimated annual water production from 2018 through 2020. In recent years, the water meter located at the WTP on the pipeline that conveys treated water to the City has been out of service. Therefore, the raw water intake volume from the Columbia River from 2018 through 2020 was compared with historical records from the WTP (from 2003 through 2006) to approximate the percentage of raw water that is delivered to the City system. Based on the historical treatment plant records, it was assumed that approximately 88 percent of raw water is delivered to the distribution system as potable water.

Table 2-1. Historical Annual Water Production[(a)]

Year	Raw Water, MG		Total Production[(b)], MG	Average Day Production, gpd
	Columbia River	Fox Creek		
2018	199.2	--	176.1	482,399
2019	197.5	--	174.5	478,116
2020	185.8	--	164.2	448,683
Average	**194.2**	**--**	**171.6**	**469,732**

(a) Historical production was unavailable due to an out of service meter at the WTP. Therefore, the raw water intake volumes were used to approximate the total water produced at the WTP for delivery to the City's water distribution system.

(b) Total production shown was approximated based on historical data. Upon review of historical data from 2003 to 2006, it was determined that approximately 88.4 percent of raw water is produced for delivery to the City as potable water.

MG = Million Gallons

gpd = Gallon Per Day

As discussed previously, the City has relied solely on Columbia River raw water since 2009. Water production in recent years has been reduced due to efforts by the City to limit wasting water, such as process control changes at the WTP and leak repair in the City's public park.

2.3.2 Water Consumption

Table 2-2 summarizes the historical water consumption from 2017 through 2020, developed from meter data provided by the City. The City does not categorize billing records by customer type, but a majority of customer connections are residential. Historically, United States Gypsum (USG) has been the City's single largest water user. From 2018 through 2020, USG accounted for approximately 51 percent of the City's total consumption.

Table 2-2. Historical Metered Water Consumption

Year	Number of Metered Service Connections	Large User - USG[a]		Total Metered Consumption	
		Total Annual Demand, MG	Percent of Total	Total Annual Demand, MG	Average Daily Demand, gpd
2017	843	0.0	0	61.3	167,806
2018	862	77.7	55	142.5	390,381
2019	890	73.1	52	139.8	382,978
2020	958	68.3	46	148.4	405,486
Average		--	--	**123.0**	**336,663**
Average (2018-2020)[b]		**73.0**	51	**143.6**	**392,948**

Source: City metered billing records, 2018-2020

(a) USG demand based on historical records for Meter IDs 10 0120, 10 0130, and 10 0135.

(b) Average consumption is based on 2018 to 2020 data. Annual consumption for 2017 is considered an outlier due to lack of water consumption at USG in 2017.

2.3.3 Non-Revenue Water

NRW is the difference between the quantity of water produced and the quantity of water consumed or metered. There are various reasons why the total customer water use is less than the total amount of water produced by the City. The most common reasons for NRW are due to system losses such as leakage, errors in measurement, and unmetered or unauthorized use. While water providers strive to minimize the amount of NRW, it is difficult to eliminate entirely.

Table 2-3 summarizes the City's NRW from 2018 through 2020. There is a noticeable drop in NRW from 2019 to 2020 that can be attributed to the City's deliberate efforts to reduce water use at the WTP and other City-owned properties, as well as an increase in the number of meters read by the City. Several meters in the City's public park and wastewater treatment plant were not read regularly until 2019 and 2020.5 While NRW rates are historically higher than is typically desired (i.e., below 10 percent), the NRW percentage is decreasing due to past and current concerted improvement efforts by City staff. For planning purposes,

[5] Per email correspondence with City staff.

the 2020 NRW percentage of approximately 10 percent is recommended to calculate the City's total future water production requirement.

Table 2-3. Historical Non-Revenue Water

Year	Total Production, MG	Total Consumption, MG	Water Loss, MG	Non-Revenue Water, percent
2018	176.1	142.5	33.6	19.1
2019	174.5	139.8	34.7	19.9
2020	164.2	148.4	15.8	9.6
Average	171.6	143.6	28.0	16.2

Source: Refer to Table 2-1 and Table 2-2.

2.3.4 Peaking Factors

Accurate peak demands are critical for evaluating and sizing water system transmission/distribution pipelines and storage facilities and defining water supply needs and capacity requirements. Projecting peak demands typically involves applying a multiplier, or peaking factor, to the average day demand. An average day demand for a particular year is calculated by taking the total annual water production divided by the total number of days in that year (refer to Table 2-1).

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

2.3.4.1 Maximum Day Demand Peaking Factor

The MDD peaking factor is calculated by dividing the calendar year's largest, single-day demand by the average day demand (ADD) of the same year. Separate MDD peaking factors were evaluated for USG and the remainder of the City, as USG accounts for approximately half of the metered water consumption and the seasonal and daily water use pattern of an industrial customer can vary significantly from typical residential water use. The MDD peaking factor for USG was approximated based on the maximum month consumption, as provided in the monthly metered demand data provided by the City. Table 2-4 shows the ADD, maximum month demand, and maximum month demand peaking factor for USG for 2018 through 2020. For planning purposes, the average 2019 through 2020 maximum month peaking factor of 1.2 times the ADD is recommended for use as USG's MDD peaking factor.

Table 2-4. Historical Maximum Month Demand Peaking Factors – USG

Year	ADD[a], gpd	Maximum Month Demand[b], gpd	Maximum Month Peaking Factor
2018	212,776	306,721	1.44
2019	200,147	249,964	1.25
2020	186,549	217,020	1.16
Average	199,824	**257,902**	**1.28**
Average (2019-2020)	193,348	**233,492**	**1.21**

(a) Refer to Table 2-2.
(b) Maximum month demand calculated from City billing data.

As shown in Table 2-5, the City's maximum day peaking factors from 2018 through 2020 range from 2.34 (2020) to 3.48 (2018), with an average of 3.02. In 2020, the City began consistently recording production values at a similar time each day. The City's production has been historically recorded daily but only represents a true 24-hour production when the data is recorded in approximate 24-hour intervals. In addition, the long-term impacts of the COVID-19 pandemic on community water consumption patterns is largely unknown. Consequently, for planning purposes in this 2021 WMP update, the average 2019 to 2020 MDD peaking factor of 2.8 times the ADD is recommended.

Table 2-5. Historical Maximum Day Demand Peaking Factors - City

	Average Day Production, gpd			Maximum Day Production, gpd			MDD Peaking Factor - City
Year	USG[a]	City[b]	Total[c]	USG[a]	City[b]	Total[d]	
2018	236,418	245,981	482,399	340,801	856,613	1,197,414	3.48
2019	222,386	255,730	478,116	277,738	832,189	1,109,927	3.25
2020	207,277	241,406	448,683	241,133	563,918	805,051	2.34
Average	**222,027**	**247,705**	**469,732**	**286,557**	**750,907**	**1,037,464**	**3.02**
Average (2019-2020)	**214,831**	**248,568**	**463,399**	**259,436**	**698,053**	**957,489**	**2.80**

(a) WTP production attributed to USG is based on the metered consumption data (see Table 2-4) with an applied NRW of 10 percent.
(b) WTP production attributed to City demand = Total - USG
(c) See Table 2-1.
(d) Maximum day raw water intake data provided by City. Maximum day production values shown assume that approximately 88.4 percent of the raw water is produced for delivery to the City as potable water.

2.3.4.2 Peak Hour Demand Peaking Factor

The PHD peaking factor is typically calculated by dividing the calendar year's largest single hour demand by the ADD of the same year. However, the flow meter from the WTP into the distribution system is out of service and the City does not have sufficient records to calculate the PHD based on historical data. Instead, a review of other Western Oregon communities with similar climate and variation in seasonal demand indicates that a PHD factor of 1.5 times the MDD is appropriate for planning purposes. Using this method, a PHD of 4.2 times the ADD is assumed for this WMP Update.

2.3.4.3 Recommended Peaking Factors

Based on the analysis in Section 2.3.4 above, the peaking factors presented in Table 2-6 are recommended for planning purposes. USG is assumed to require a constant water demand over a 24-hour period to maintain operations, which is reflected in the equal MDD and PHD peaking factors of 1.2 times the ADD.

Table 2-6. Water Demand Peaking Factors

Demand Condition	Peaking Factor	
	USG	City
Average Day Demand	1.00	1.00
Maximum Day Demand	1.2 x ADD	2.8 x ADD
Peak Hour Demand	1.2 x ADD	4.2 x ADD

2.4 EXISTING WATER SYSTEM FACILITIES

The City's key water system facilities are shown on Figure 2-2 and discussed in more detail in the sections below. Figure 2-2 shows a plan view of the City's distribution system and key water system facilities. The evaluation of facility capacities and their ability to meet future water demands is described in *Chapter 4 Water System Analysis*.

2.4.1 Existing Water Treatment Facilities

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

2.4.1.1 Fox Creek Impoundment Raw Water Intake

The City impounds water from Fox Creek at the Fox Creek Reservoir, a small timber dam and reservoir constructed in 1910 at an approximate elevation of 350 feet. The dam's maximum height is 25 feet, with a maximum water depth of 22 feet in the reservoir. A 36-inch by 36-inch lift-type sluice outlet gate allows water to flow through a 1.56-mile 6-inch pipeline to the WTP. A portion of the 6-inch CI raw water pipeline was destroyed in a washout and replaced with 6-inch HDPE pipeline in 2016.

2.4.1.2 Columbia River Raw Water Intake

The City pumps Columbia River raw water from a 32-foot deep wet well and pump station located on the bank of the river. A 12-inch raw water intake set at an elevation of -19 feet deep in the Columbia River delivers water to the wet well, where it is pumped uphill to the water treatment plant by two pumps, set at an elevation of 24 feet, through a 10-inch cast iron pipeline. Each Columbia River raw water pump is a 100 horsepower (hp) pump with a design capacity of 825 gpm at 320 feet of total dynamic head (TDH).

The Columbia River intake was built in 1968 and retrofitted in 2000, when the pumps and housing structure were replaced.

2.4.1.3 Water Treatment Plant

Raw water from Fox Creek and the Columbia River is conveyed through separate pipelines to the City-owned and operated WTP. As previously discussed, water is pumped from the Columbia River and gravity-fed from the Fox Creek Reservoir. Prior to 2009, the City typically blended no more than 30 percent of the total supply as Fox Creek raw water. The City has since shut down the Fox Creek intake due to water quality concerns and is treating solely Columbia River raw water.

The WTP was constructed new in two phases in 2000 and 2001. Raw water is treated through the City's water treatment facilities, including a chemical feed system (currently disabled), mechanical mixers, backwash waste lagoon, clarifiers, filters, and chemical disinfection. The treated and disinfected water is then stored in a 0.5 MG baffled clearwell (WTP clearwell). An on-site pump station (further described in Section 2.4.2.4) delivers the finished water to the Neer City Reservoir and the upper-level distribution system, while the remaining system is supplied from the clearwell by gravity. A 2018 water system survey conducted by the Oregon Health Authority (OHA) concluded that the WTP has a maximum total rated flow capacity of 1,400 gpm.

N

0 600 1,200

Scale in Feet

Symbology

- Storage Reservoir
- Pump Station
- Water Treatment Plant
- Raw Water Intake
- Pressure Regulating Valve
- Pressure Regulating/Sustaining Valve
- Parcels

Pressure Zone

- Main-High
- Main-Low
- Old Highway
- Townsend
- Upper

Potable Water Pipelines

- Less than 8-inches
- Greater than 8-inches

Raw Water Pipelines

- Columbia River
- Fox Creek

Old Hwy, 0.75 MG

Old Highway PS

Townsend PS

Townsend, 0.015 MG

Clearwell, 0.5 MG

WTP High Service PS

Neer City, 0.5 MG

Columbia River

Rinearson Slough

Nice Creek

Fox Creek

Owl Creek

Beaver Creek

WEST YOST - N:\Clients\962 City of Rainier\60-20-01 Water MP Update\GIS\MXD\Report\Fig2-2 ExSystem.mxd - nsziganti - 3/22/2023

Figure 2-2

Existing Water System

City of Rainier
Water Master Plan Update

2.4.2 Existing Water Distribution System

The existing water distribution system includes:

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

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

2.4.2.1 Pressure Zones

The City operates a total of five (5) pressure, outlined in Figure 2-2, with one storage facility serving each zone. Zone-specific information is shown in Table 2-7. A majority of the City's service connections are located in the Main-High Zone, which is gravity-fed directly from the WTP clearwell. The Main-Low zone primarily serves industrial customers, and is fed by the Main-High Zone and the Old Highway Reservoir. The Upper Zone is served by pumps at the WTP and the Neer City Reservoir, and the Townsend Zone is served by the Townsend Buried Pump Station and the Townsend Reservoir. The Old Highway Zone is fed by a hydropneumatic tank located at the Old Highway Pump Station, which draws from the Old Highway Reservoir. A control valve located at Washington Way and Old Rainier Road operates to fill the Old Highway Reservoir and regulates flows from the Main-High Zone to the Main-Low Zone. Service connections located along Old Rainier Road are sited at a higher elevation than the Old Highway Reservoir and are maintained at adequate service pressures by the hydropneumatic tank, which maintains pressures between 59 and 80 psi.

Table 2-7. Summary of Existing Pressure Zones

Zone Name	Existing Minimum Service Elevation, ft[(a)]	Existing Maximum Service Elevation, ft[(a)]	Static Pressure Range, psi
Main-High	16	272	18 - 109
Main-Low	12	184	18 - 81
Upper	36	404	33 - 156
Old Highway	131	272	59 - 80
Townsend	262	394	35 - 178

(a) Service elevation is approximate based on 2009 USGS DEM for the Oregon Northern Coast.

2.4.2.2 Distribution Mains

Table 2-8 and Table 2-9 summarize the City's existing pipelines by diameter and material type, respectively. The City's existing water system consists of approximately 18.2 miles of water system pipelines, with distribution pipelines sizes generally ranging from 1 to 8-inches in diameter. Larger transmission mains range from 10 to 16-inches in diameter, with 12-inch pipelines comprising nearly 91 percent of pipelines in that range. As shown in Table 2-8, approximately 61 percent (or 11.0 miles) of

the City's pipelines are distribution mains consisting of pipelines 2 to 8-inches in diameter. The majority of the City's pipelines—approximately 60 percent (or 10.9 miles)—are CI.

Table 2-8. Summary of Existing Raw Water Pipelines by Diameter

Pipe Diameter, inches	Length of Pipelines, feet	Length of Pipelines, miles	Percent in Water System
Unknown	946	0.18	1.0
1	5,389	1.02	5.6
2	11,331	2.15	11.8
4	12,434	2.35	13.0
6	24,410	4.62	25.4
8	10,048	1.90	10.5
10	2,358	0.45	2.5
12	28,445	5.39	29.6
16	608	0.12	0.6
Totals	**95,969**	**18.18**	**100%**

Source: City GIS pipelines shapefile (waterlines.shp), provided 12/3/20 and updated with improvements along A Street.

Table 2-9. Summary of Existing Raw Water Pipelines by Material

Pipe Material	Acronym	Length of Pipelines, feet	Length of Pipelines, miles	Percent in Water System
Asbestos Cement	AC	5,968	1.13	6.2
Cast Iron	CI	57,357	10.86	59.8
Copper	COP	114	0.02	0.1
Ductile Iron	DI	2,405	0.46	2.5
Galvanized	GALV	10,706	2.03	11.2
Polyvinyl Chloride	PVC	6,147	1.16	6.4
Steel	STL	5,005	0.95	5.2
Unknown	UNK	8,267	1.57	8.6
Totals		**95,969**	**18.18**	**100%**

Source: City GIS pipelines shapefile (waterlines.shp), provided 12/3/20 and updated with improvements along A Street.

2.4.2.3 Storage Facilities

The City has four (4) storage reservoirs within its water service area, with a total storage capacity of 1.765 MG. The location of each reservoir is shown on Figure 2-2, with key information for each facility shown in Table 2-10. Three (3) of the City's storage reservoirs are sited at an elevation that establishes the hydraulic grade for the pressure zone, which allows the reservoir to supply the system by gravity.

The Old Highway Reservoir is located in the Main Zone but is sited at a lower elevation than the hydraulic grade established by the WTP clearwell. Therefore, the 12-inch control valve located at Washington Way and Old Rainier Road functions as the Old Highway Reservoir's altitude valve and operates to control flow into the lower Main Zone. When the control valve is open, the Old Highway Reservoir fills due to the higher hydraulic grade, driven by the elevation of the WTP clearwell; however, when the control valve is throttled or closed, the Old Highway Reservoir is forced to turn over. The Old Highway Reservoir site also contains a hydropneumatic tank in an attached pump station which serves the residential connections located above the site. The City has indicated that the existing hydropneumatic tank is old and problematic; a replacement has been budgeted for under the proposed CIP presented in *Chapter 6 Water System Capital Improvement Program*.

Apart from the WTP Clearwell, reservoirs are filled by pump stations.

Table 2-10. Summary of Existing Potable Water Storage

Facility Name	Zone	Storage Capacity, MG	Diameter, ft	Base Elevation, ft	Overflow Elevation, ft
WTP Clearwell[(a)]	Main	0.5	60	240.0	263.5
Neer City Reservoir[(a)]	Upper	0.5	54	573.0	601.5
Old Highway Reservoir[(b)]	Main / Old Highway	0.75	60	181.0	217.2
Townsend Reservoir[(c)]	Townsend	0.015	11	493.0	508.0
Total Capacity		**1.765**	-	-	-

(a) Based on the Phase I Water System Improvements Project, dated June 2000.
(b) Based on the Phase II Water System Improvements Project, dated June 2001.
(c) Based on City input, 1/27/2021. Base elevation is approximate based on 2009 USGS DEM for the Oregon Northern Coast.

2.4.2.4 Pump Stations

The City currently operates three (3) pump stations within its water service area. Each pump station is used to fill tanks and serve connections at higher elevations within each zone. Pump station locations are shown on Figure 2-2. Each pump station contains two or three pumps. Where multiple pump units are available, one pump typically is reserved as a standby unit.

The total existing pump station capacity is 600 gpm (0.86 mgd). The total existing firm capacity, with the largest pump reserved as a standby unit at each pump station is 300 gpm (0.43 mgd). Table 2-11 summarizes the key characteristics of the City's existing booster pump stations.

Table 2-11. Summary of Existing Potable Water Pump Stations

Pump Station (Zone)	Location	Pump ID	hp	Design Flow, gpm	TDH, ft	Total Pumping Capacity, gpm	Firm Pumping Capacity, gpm
WTP High Service (Main)[a]	Water Treatment Plant	P602	40	150	400	300	150
		P603	40	150	400		
Old Highway (Old Hwy)[b,c,d]	On-site at Old Highway Reservoir	Unknown	7.5	80	180	160	80
		Unknown	7.5	80	180		
		Fire Pump	25	Unknown	Unknown		
Townsend Buried (Townsend)[d]	West C Street & W 8th Street	Unknown	Unknown	70	310	140	70
		Unknown	Unknown	70	310		

(a) Based on the Phase I Water System Improvements Project, dated June 2000.
(b) Horsepower and design flow information based on City input, 1/27/2021.
(c) Fire pump not considered in calculation of total and firm pumping capacity.
(d) Information assumed based on design points provided in the City's 1995 Water Master Plan for the pump station.
hp = Horsepower

2.4.2.5 Pressure Regulating Stations

The City's water distribution system includes five (5) pressure regulating stations (PRSs) or pressure regulating valves (PRVs) that regulate the transfer of water between pressure zones in the system. Three PRSs are located in the vicinity of the WTP and regulate pressure from the Upper Zone to the Main Zone. One PRV is located in the vault sited at the intersection of Washington Way and Old Rainier Road which also contains a 12-inch control valve that functions as the altitude valve for the Old Highway Reservoir. The other PRS regulates pressure from the high elevations of the Townsend Zone into the Main Zone. The City's PRSs are shown on Figure 2-2. Table 2-12 summarizes the key characteristics of the City's existing PRSs.

Table 2-12. Summary of Existing Pressure Regulating Stations

Location Description	Serial No.	Pressure Zone Upstream / Downstream	Valve Diameter, in	Elevation[a], ft	Pressure Setting, psi	
					Upstream	Downstream
Washington & Old Rainier	92EG-04ABC	Old Highway / Main	12	124.0	60	38
						52 sustaining
Rainier & Norwood	90G-01AS	Upper / Upper[b]	2	231.9	160	90
	90G-01ABCS		6		160	85
2nd & Norwood	90G-01AS	Upper / Main	1.5	157.7	112	35 back 45
	90G-01ABCS		4		112	30 back 45
Behind WTP	90G-01AS	Upper / Upper[b]	2	274.2	145	45
	690G-01ABCS		6		145	40
Fern Hill Rd 75442	92G-01BSV	Townsend / Main	2.5	254.5	128	CRL 60 CRD 22

Source: PRV Settings.doc, provided by the City on 2/25/2021.

(a) Elevation is approximate based on 2009 USGS DEM for the Oregon Northern Coast, and a depth below grade of 3 ft.

(b) Valves reduce head from the Neer City reservoir but are not considered full zone breaks.

2.5 OPERATIONS AND MAINTENANCE OVERVIEW

The City has existing Operation and Maintenance (O&M) programs and has identified several additional programs which it plans to implement to extend the useful life of its assets, identify deficiencies, and upgrade aging infrastructure. These programs are summarized as follows:

- **Regulatory Water Quality Testing:** The City maintains an E. coli sampling and water quality sampling and analysis plan and regularly tests water quality at specific locations throughout the service area to demonstrate compliance with state and federal regulations.
- **Meter Replacement Program:** The City recently completed a full replacement of all meters in its water service area. This is part of a concerted effort to reduce non-revenue water and monitor all known water use within the service area.
- **Hydrant Flushing Program (Future):** The City does not currently maintain a hydrant flushing program but has identified hydrant flushing as a future preventative O&M and asset management program to implement. It is recommended that the City develop a Hydrant Flushing Standard Operating Procedure (SOP).
- **Hydrant Testing and Maintenance Program (Future):** The City does not currently maintain a hydrant testing and maintenance program but has identified this program as a future preventative asset management program to implement, with a goal of testing hydrants at predetermined intervals to service hydrants as needed. It is recommended that the City develop a Hydrant Testing and Maintenance SOP.
- **Valve Exercising Program (Future):** The City does not currently maintain a valve exercising program but has identified this program as a future preventative asset management program to implement, with a goal of operating water system valves on predetermined intervals to extend the useful life and track the condition of the City's valves. It is recommended that the City develop a Valve Exercising SOP.

CHAPTER 3
Water System Performance Criteria

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

Key water system planning criteria from the 1995 WMP have been incorporated into this chapter. Some of the previous criteria have been revised to reflect more recent or suitable water system industry standards for this 2021 WMP Update, including standards from the Oregon Drinking Water Services (DWS), the Environmental Protection Agency (EPA), the American Water Works Association (AWWA), and the Oregon Fire Code (OFC). The following sections of this chapter present the recommended planning criteria for the City's water distribution system:

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

Table 3-1 summarizes the recommended water system planning criteria for the 2021 WMP Update, which are discussed in more detail in the section below.

Table 3-1. City of Rainier Planning and Design Criteria

Component	Criteria	Remarks / Issues
Fire Flow Requirement		
Residential	1,000 gpm @ 1 hour	
School	3,000 gpm @ 3 hours	
Commercial	3,000 gpm @ 3 hours	
Industrial	4,000 gpm @ 4 hours	
Water Supply Capacity		
Supply/Pumping Capacity	Provide capacity equal to maximum day demand	
Pumping Facility Capacity		
Pumping Capacity	Provide capacity equal to maximum day demand for the pressure zone	Design for peak hour or maximum day demand plus fire flow (whichever is larger), only if no gravity storage is available within the pressure zone.
Water Storage Capacity		
Operational Storage	30 percent of maximum day demand	
Fire Storage	Varies dependent upon fire flow duration	1,000 gpm @ 1 hour = 0.06 MG
		3,000 gpm @ 3 hours = 0.54 MG
Emergency Storage	100 percent of average day demand	
Total Water Storage Capacity	Operational + Fire + Emergency	
Pipeline Sizing		
Diameter - Transmission	12-inches or larger	
Diameter - Distribution	Less than 12-inches	
Maximum Pressure (psi)	100	According to the Uniform Plumbing Code, residences with pressures above 80 psi must have pressure reducing valves.
Minimum Pressure (psi)		
Average Day Demand	45	
Maximum Day Demand	45	
Maximum Day Demand plus Fire Flow	20	
Peak Hour Demand	35	
Maximum Pipeline Velocity (fps)		
Average Day Demand	5	New pipelines only.
Maximum Day Demand	5	New pipelines only.
Maximum Day Demand plus Fire Flow	12	New pipelines only.
Peak Hour Demand	5	New pipelines only.
Pipeline Material	DIP; PVC	
Hazen Williams "C" Factor	120 (DIP); 130 (PVC)	For consistency in hydraulic modeling.

3.1 GENERAL WATER SYSTEM RECOMMENDATIONS

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

3.1.1 Water Quality Standards

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

3.1.2 Water System Reliability

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

3.2 WATER SYSTEM CAPACITY AND PERFORMANCE

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

3.2.1 Fire Flow Requirements

Fire flow requirements were established in the 1995 WMP by the City of Rainier Fire Chief. The City's minimum fire flow requirements are generally consistent with the 2019 Oregon Fire Code Tables B105.1 and B105.2, which establish minimum fire flows and durations for individual structures based on the structure's construction type and fire flow calculation area. The fire flow requirements presented in this 2021 WMP Update have not been developed for specific structures and are intended only for general planning purposes.

Table 3-2 summarizes the recommended minimum fire flow requirements by general land use designations. Fire flows shall be met concurrently with a maximum day demand condition, while maintaining a minimum distribution system residual pressure of 20 pounds per square inch (psi). Fire flows and the expected duration will also be used to establish treated water storage requirements.

Table 3-2. Fire Flow Requirements

Land Use Designation[a]	Required Fire Flow, gpm	Duration, hours
Residential (SR, R-1, R-2, R-3)	1,000	1
School[b]	3,000	3
Commercial (C-1, C-2, WC, WM, WMO, CBD)	3,000	3
Industrial (M-1, M-2)[c]	4,000	4

(a) Land use designations are based on the City of Rainier Zoning Ordinance, amended 7/17/2017.

(b) Schools are located within the R-2 and R-3 zoning districts. A more stringent fire flow requirement will be assigned to schools, regardless of the land use designation.

(c) Industrial fire flow requirements may be met by a combination of City water and/or Columbia River raw water, provided that either the industrial customer or the City of Rainier Fire Department is equipped with an appropriately sized raw water fire pump.

3.2.2 Water Supply and Treatment Capacity

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

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

3.2.3 Distribution System Pressures

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

- Allowable Pressures Under Normal Operating Conditions: 45 psi to 100 psi[1]
  - Minimum Pressure under Average Day Demand: 45 psi
  - Minimum Pressure under Maximum Day Demand: 45 psi
  - Minimum Pressure under Peak Hour Demand: 35 psi
- Minimum Pressure Under Fire Flow Conditions: 20 psi

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

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

3.3 FACILITIES SIZING

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

3.3.1 Pumping Facility Capacity

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

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

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

3.3.2 Water Storage Capacity

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

- Operational Storage
- Fire Storage
- Emergency Storage

Each storage component is discussed below. The recommended water storage capacity for the City's water system will be evaluated by pressure zone.

3.3.2.1 Operational Storage

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

Typically, water treatment plants are operated at a constant rate over a 24-hour period (baseline). Additional flow is provided from storage tanks during peak demand periods. Storage tanks are typically replenished when demands drop below the baseline water supply. Supply is typically provided at a rate equal to maximum day demand. The storage volume used to meet the peak demand periods is called operational storage.

For a typical system, the volume of water recommended to be held in reserve for operational flow should be at least equal to 30 percent of the total volume of water used on the maximum day.[2]

3.3.2.2 Fire Storage

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

3.3.2.3 Emergency Storage

A storage reserve is required to meet demands during an emergency. An emergency is defined as an unforeseen or unplanned event that may degrade the quality or quantity of potable water supplies available to serve customers. Determination of the required volume of emergency storage is a policy discussion based on the assessment of the risk of failures and the desired degree of system reliability. The amount of required emergency storage is a function of several factors including the diversity of the supply sources, redundancy and reliability of the production facilities, and the anticipated length of the emergency outage.

The AWWA states that no formula exists for determining the amount of emergency storage required, and that the decision will be made by the individual utility based on a judgment about the perceived vulnerability of the system. For this 2021 WMP Update, it is recommended that the City have a minimum quantity of emergency storage volume equivalent to 100 percent of the average day demand.

3.3.2.4 Total Storage Capacity Recommended

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

- **Operational:** Volume of water necessary to meet diurnal peaks observed throughout the day, assumed to be equivalent to at least 30 percent of the maximum day demand; plus
- **Fire Flow**: Volume of water necessary to supply a fire flow event, where the fire flow event is contingent upon the land use designation; plus
- **Emergency**: Volume of water necessary to provide an emergency supply of 100 percent of the average day demand.

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

3.3.3 Pipeline Sizing

The following criteria will be used as guidelines for sizing transmission and distribution system pipelines. Although these criteria and guidelines have been established and will be used to size new pipelines, the City's existing water system should be evaluated using system pressure as the primary criterion.

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

Secondary criteria, such as pipeline velocity, head loss, age, and material type, are used as indicators to locate, and to help prioritize where water system improvements may be needed.

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

3.3.3.1 General Definitions and Standards

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

- Transmission pipelines are generally 12-inches in diameter.
- Distribution pipelines are generally less than 12-inches in diameter.
- All new pipelines are required to be ductile iron pipe (DIP) or PVC.
- All new pipelines are required to have a minimum diameter of 8-inches, or 6-inches for dead-end mains only.

3.3.3.2 Average Day Demand

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

- Pressures should be maintained between 45 and 100 psi at the customer service elevation. According to the Uniform Plumbing Code, residences with pressures above 80 psi must have pressure reducing valves.
- The maximum velocity within new pipelines should be 5 feet per second (fps).

3.3.3.3 Maximum Day Demand

West Yost recommends evaluating maximum day demand conditions using the recommendations listed in *Section 3.3.3.2 Average Day Demand*.

3.3.3.4 Maximum Day Demand plus Fire Flow

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

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

3.3.3.5 Peak Hour Demand

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

- The minimum residual pressure during a peak hour demand should be 35 psi.
- The maximum velocity within new pipelines should be 5 fps.

CHAPTER 4
Water System Analysis

This chapter presents an analysis of the City's existing water system and its ability to meet recommended water service and performance standards under a projected future 20-year growth master plan horizon. The analysis includes both system capacity and hydraulic performance evaluations based on the performance criteria presented in *Chapter 3 Water System Performance Criteria*. The system capacity evaluation includes an evaluation of supply, pumping, and storage capacity. The performance evaluation assesses the water system's ability to meet recommended performance standards under maximum day demand plus fire flow and peak hour demand conditions.

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

- Water Demand
- Water System Capacity Evaluation
- Water System Performance Evaluation
- Summary of Findings and Recommendations

4.1 WATER DEMAND

The water demands used for the water system evaluation represent the existing demands plus projected growth over the 20-year master plan timeframe. Existing water demands were determined in *Chapter 2 Existing Water System*. For planning purposes, the 2019 average day demand of approximately 478,100 gpd is recommended as a baseline existing demand due to possible impacts on annual water demand from local shutdowns influenced by the 2020 COVID-19 pandemic. The City expects very low growth over the 20-year master plan horizon; therefore, a uniform 0.5 percent annual increase in water demand from the baseline demand will be evaluated across the entire City water system.

The projected water demand used in this analysis is a total system average day demand of approximately 528,000 gpd. This equates to a 10.5 percent increase in demand from 2020 to 2040. Maximum day and peak hour demands were calculated based on the adopted City peaking factors of 2.8 and 4.2 times the average day demand, respectively, as described in *Chapter 2 Existing Water System.* Additionally, the peaking factors for industrial customer USG remain constant at 1.2 times the average day demand for maximum day and peak hour demands. The water demands used for the water system evaluation are summarized by pressure zone in Table 4-1.

Table 4-1. Summary of Projected Water Demands by Pressure Zone

Pressure Zone	Average Day Demand[a]		Maximum Day Demand[b]		Peak Hour Demand[c]	
	gpm	gpd	gpm	gpd	gpm	gpd
Main-High	129.1	185,850	361.4	520,380	542.1	780,570
Main-Low	189.2	272,510	256.3	369,080	281.9	405,890
Upper	32.6	46,950	91.3	131,460	136.9	197,200
Old Highway	4.3	6,220	12.1	17,430	18.2	26,140
Townsend	11.6	16,670	32.4	46,680	48.6	70,020
Total (USG)	171.0	246,210	205.2	295,450	205.2	295,450
Total (City)	195.8	281,990	548.3	789,580	822.5	1,184,370
Total	**367**	**528,200**	**754**	**1,085,030**	**1,028**	**1,479,820**

(a) Existing demands are based on 2019 water production. Historical Metered Billing data provided by the City in February 2021.Projected water demands assume 10.5 percent growth in demand from existing conditions. Demands by zone based on demands allocated to the hydraulic model (in gpm). Therefore, total demand (in gpd) may not round to the nearest gpd if scaled using demands shown in Table 2-5.

(b) Maximum day demand calculated using a peaking factor of 2.8 times the average day demand (USG peaking factor = 1.2).

(c) Peak hour demand calculated using a peaking factor of 4.2 times the average day demand (USG peaking factor = 1.2).

4.2 WATER SYSTEM CAPACITY EVALUATION

To evaluate the capacity of the City's water system, the following analyses were conducted:

- Water Supply Capacity Evaluation
- Pumping Capacity Evaluation
- Storage Capacity Evaluation

4.2.1 Water Supply Capacity Evaluation

The City's water supply is provided by local surface water diverted from the Columbia River and Fox Creek and treated at the City's water treatment plant, as described in *Chapter 2 Existing Water System.* The City's water supply and treatment capacity criterion requires the City to produce sufficient supply to serve a maximum day demand condition. Because the City controls the diversion and treatment of the potable water it delivers to customers, all stages of the treatment process must be capable of meeting the water supply criterion. The following sections evaluate the supply capacity of all diversion and treatment facilities:

- Water Rights Capacity
- Raw Water Intake Capacity
- Water Treatment Capacity
- Summary of Supply Capacity Evaluation

4.2.1.1 Water Rights Capacity Evaluation

The City holds existing water rights to divert 2.0 mgd from the Columbia River and 0.81 mgd of water from Fox Creek. The total existing water rights of 2.81 mgd available to the City are summarized in Table 4-2. This supply capacity is equal to over 250 percent of the City's future maximum day demand of 1.09 mgd and is sufficient to meet the City's supply capacity criterion.

Table 4-2. Available Water Rights versus Required Supply Capacity

Supply Source	Maximum Water Supply Capacity	
	gpm	mgd
Columbia River[(a)]	1,391	2.00
Fox Creek[(b)]	561	0.81
Total	**1,952**	**2.81**
Required Supply Capacity[(c)]	754	1.09
Columbia River Only - Existing Supply Capacity Surplus (Deficit)	**637**	**0.92**
Total Existing Supply Capacity Surplus (Deficit)	**1,198**	**1.73**

(a) Columbia River supply capacity is based on Oregon Water Resources Department Permit No. 33386.
(b) Fox Creek supply capacity is based on two Oregon Water Resources Department Permits, Nos. 21029 and 34978. Fox Creek supply can currently be utilized for emergency use only.
(c) Required supply capacity is equal to the projected maximum day demand (see Table 4-1).

4.2.1.2 Raw Water Intake Capacity Evaluation

As discussed previously, the City diverts water from the Columbia River and Fox Creek. The raw water pipelines and Columbia River supply intake location are shown on Figure 2-2.

The Columbia River intake consists of two 825 gpm (1.19 mgd) raw water pumps that discharge into a 10-inch diameter ductile iron raw water transmission pipeline. The Columbia River raw water pump station is capable of supplying 825 gpm at firm capacity, shown in Table 4-3, which can be conveyed through the intake pipeline. Under the maximum velocity constraint provided in *Chapter 3 Water System Performance Criteria*, the Columbia River raw water conveyance pipeline can supply a maximum of 1,224 gpm (1.76 mgd).

The Fox Creek intake consists of a gravity-fed 6-inch CI and HDPE pipeline from the Fox Creek Reservoir. Under the maximum velocity constraint provided in *Chapter 3 Water System Performance Criteria*, the Fox Creek raw water conveyance pipeline can supply a maximum of 441 gpm (0.63 mgd).

As discussed previously, the City's water supply criterion requires the City to provide a sufficient supply capacity equal to the maximum day demand of 1.09 mgd. Table 4-3 summarizes the total supply capacity of 1.82 mgd provided by the raw water pumping and intake facilities, which is sufficient to meet the City's supply capacity requirements.

Table 4-3. Available Raw Water Intake Facility Capacity versus Required Supply Capacity

Supply Source	Intake Pipeline Diameter	Maximum Raw Water Intake Pipeline Capacity[a]	
		gpm	mgd
Columbia River			
Raw Water Pumps[b] – Firm Capacity	--	825	1.19
Intake Pipeline	10	1,224	1.76
Fox Creek			
Intake Pipeline	6	441	0.63
Total Raw Water Intake Capacity[c]	--	1,266	1.82
Required Supply Capacity	--	754	1.09
Columbia River Only - Existing Supply Capacity Surplus (Deficit)	--	**71**	**0.10**
Total Existing Supply Capacity Surplus (Deficit)	--	**512**	**0.73**

(a) The maximum raw water intake pipeline capacity assumes the pipeline velocity does not exceed the City's maximum pipeline velocity of 5 fps, as defined in *Chapter 3 Water System Performance Criteria*.

(b) The raw water intake consists of two 825 gpm pumps that are assumed to operate at firm capacity.

(c) Total raw water intake capacity is equal to the sum of the minimum capacity available from the Columbia River raw water intake (raw water pumps or intake pipeline) and the capacity available from Fox Creek.

The City currently relies on the Columbia River intake facility exclusively, reserving the Fox Creek intake for emergency use. There is sufficient capacity from the Columbia River raw water pumps operating at firm capacity to meet the City's supply capacity requirements without supply from Fox Creek. If the City were to utilize Fox Creek supply in the event of a Columbia supply reduction or outage, existing limitations at the WTP, as described in *Chapter 2 Existing Water System*, could interfere with the ability of the City to properly treat the Fox Creek raw water. Biweekly sampling and analysis would also be required by OHA to confirm that the Fox Creek water produced by the WTP meets acceptable regulatory water quality standards. Therefore, it is recommended that a Fox Creek water quality study be performed to identify an approach to blend and treat the Fox Creek source water with supply from the Columbia River.

4.2.1.3 Water Treatment Capacity Evaluation

As presented in *Chapter 2 Existing Water* System, the City's water treatment plant has three water treatment units for a total maximum water treatment capacity of 1,400 gpm (2.0 mgd). As shown in Table 4-4, the City's treatment capacity available at the water treatment plant can supply the projected maximum day demand of 1.09 mgd. Therefore, no improvements are recommended to increase water treatment capacity.

Table 4-4. Available Water Treatment Facility Capacity versus Required Supply Capacity

Water Treatment Component	Maximum Water Treatment Component Capacity[a]	
	gpm	mgd
Treatment Unit #1	350	0.50
Treatment Unit #2	350	0.50
Treatment Unit #3	700	1.01
Total Treatment Capacity	**1,400**	**2.02**
Required Supply Capacity[c]	754	1.09
Total Supply Capacity Surplus (Deficit)	**646**	**0.93**

(a) Treatment unit capacity based on the *Water Treatment Plant Phase II* Improvement plans, dated November 2001.
(b) Required supply capacity is equal to the maximum day demand, as defined in *Chapter 3 Water System Performance Criteria.*

4.2.1.4 Summary of Supply Capacity Evaluation

The City's existing water supply system meets all water supply criteria outlined in *Chapter 3 Water System Performance Criteria.* The City's water rights, raw water intake, and water treatment facility are able to produce the projected maximum day demand equal to 754 gpm (1.09 mgd). The City's current operations, utilizing strictly Columbia River supply, will yield a surplus of 71 gpm (103,000 gpd) under future demand conditions and does not indicate a need to utilize Fox Creek supply at buildout. Prior to using Fox Creek, a water quality study is recommended due to historical water quality challenges. Additionally, as the WTP lacks the proper facilities to regularly treat Fox Creek water, this study should evaluate the existing WTP and determine necessary improvements that would be required to treat Fox Creek to meet regulatory standards, such as:

- Chemical feed system upgrades
- Raw water blending reservoir
- Supervisory Control and Data Acquisition (SCADA) system upgrades
- Additional monitoring programs, as required by OHA

4.2.2 Pumping Capacity Evaluation

The City currently operates three pump stations that serve to lift water into higher pressure zones. The pumping capacity criterion for the City, described previously in *Chapter 3 Water System Performance Criteria*, requires the City's water system to provide sufficient pumping capacity to meet demands during normal operating conditions. Normal operating conditions are defined as follows:

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

Firm pumping capacity assumes a reduction in total pumping capacity to account for pumps that are out of service at any given time due to mechanical breakdowns, routine maintenance, other operational problems, or water quality issues. At each pump station, firm pumping capacity is defined as the total pump station capacity with the largest pump out of service and is therefore not counted towards the overall total. Pump stations with only one pump will not be considered reliable.

Table 4-5 compares the existing firm pumping capacity to the required pumping capacity for each pressure zone. The Main Zone was excluded from this evaluation as it is served by gravity from the WTP Clearwell. The pumping capacity analysis indicates that each zone has adequate firm pumping capacity to meet the City's pumping criterion. It should be noted that a fire pump exists to serve the Old Highway Zone, but the pump capacity is unknown. If the design capacity of the fire pump is less than the required fire flow within the zone (1,000 gpm), it is recommended to replace the existing fire pump with one that can serve the maximum fire flow requirement in the pressure zone. In addition, it is recommended to replace the Old Highway hydropneumatic tank to prevent excessive wear on the Old Highway Pump Station and improve supply reliability to the Old Highway Zone. As noted in *Chapter 2 Existing Water System,* the existing hydropneumatic tank located at the Old Highway Pump Station is old and problematic.

Table 4-5. Available Pumping Capacity versus Required Pumping Capacity

Pressure Zone	Pump Station	Available Pumping Capacity, gpm		Required Pumping Capacity[a], gpm		Pumping Capacity Surplus (Deficit), gpm
		Total Capacity	Firm Capacity	Criteria	Required Capacity	
Upper	WTP Booster	300	150	MDD	91	59
Old Highway[b]	Old Highway	160	80	PHD	18	62
		Fire Pump - Unknown		MDD + Fire	1,012	Unknown
Townsend	Townsend Buried	140	70	MDD	32	38
Main-High / Main-Low	Zones were not evaluated for pumping capacity since they are gravity fed directly from the WTP Clearwell					

(a) Required pumping capacity for zones with adequate storage is equal to the maximum day demand for the pressure zone, while zones without adequate storage require pumping capacity equal to the greater of peak hour demand or maximum day demand plus fire flow, as defined in *Chapter 3 Water System* Performance Criteria. Demands by zone are shown in Table 4-1.

(b) The Old Highway pump station's fire pump capacity is unknown and is assumed to be deficient (less than 1,000 gpm).

4.2.3 Storage Reservoir Capacity Evaluation

The City has 4 storage reservoirs serving 5 pressure zones, providing a total system water storage capacity of 1.765 MG. To comply with the design and operational criteria, the water system should provide adequate operational storage to balance differences in demands and supplies, emergency storage in case of supply failure, and water to fight fires. The City's available above-ground storage (i.e., storage tanks) must be sufficient to meet the City's operational, emergency, and fire flow storage criteria.

The City's water storage capacity requirement is described in Chapter 3, and is as follows:

- Operational storage equal to 30 percent of maximum day demand
- Emergency storage equal to one average day demand
- Fire flow storage equal to the highest fire flow multiplied by the recommended duration

The City's existing water storage facilities were evaluated to determine whether the City's existing water system has sufficient storage capacity to provide the recommended operational, emergency, and fire flow storage. Table 4-6 compares the City's available water storage capacity with the required storage capacity by pressure zone. As shown, there is sufficient storage capacity available to meet the required storage capacity criteria established in Chapter 3 in all zones except the Upper Zone and the Townsend Zone. Therefore, it is recommended that the City increase the storage capacity within the Upper Zone and Townsend Zone by 0.13 MG and 0.08 MG, respectively. The existing 0.015 MG Townsend Reservoir replaced an adjacent 0.04 MG redwood tank. It is recommended that the abandoned redwood tank be demolished, and a new storage facility be constructed next to the existing Townsend Reservoir.

Table 4-6. Available Storage Capacity versus Required Storage Capacity

Pressure Zone(s)	Available Storage Capacity, MG			Required Storage Capacity, MG				Storage Surplus (Deficit)
	Storage Facility	Storage Facility Capacity	Zone Storage[a]	Operational[b]	Emergency[c]	Fire[d]	Total	
Main-High[e]	WTP Clearwell	0.50	1.02	0.21	0.25	0.54	1.00	0.02
Main-Low / Old Hwy[f]	Old Highway Reservoir	0.75	1.77	0.33	0.53	0.54	1.39	0.37
Upper	Neer City Reservoir	0.50	0.50	0.04	0.05	0.54	0.63	(0.13)
Townsend	Townsend Reservoir	0.02	0.02	0.01	0.02	0.06	0.09	(0.08)

(a) Storage available to the pressure zone consists of all storage directly serving the pressure zone, including the storage located within the zone as well as any storage located in upper pressure zones that are capable of serving the evaluated pressure zone under normal operations (e.g., through a PRV or other control valve).

(b) Operational storage capacity is equal to 30 percent of the maximum day demand of the zone and all upper zones utilized for storage (see note a). See Table 4-1 for projected maximum day demand.

(c) Emergency storage capacity is equal to one average day demand of the zone plus all upper zones. See table 4-1 for projected average day demand.

(d) Fire flow storage capacity required is equal to the largest fire flow possible in zone: 3,000 gpm for 3 hours for Main-High, Upper, and Main-Low Zones; 1,000 gpm for 1 hour for all other zones. It is assumed that the 4,000 gpm industrial fire flow demands along the waterfront can be supplied by a combination of treated City water and Columbia River raw water.

(e) The Main-High Zone was evaluated assuming storage is available for use in the WTP Clearwell, Townsend Reservoir, and Neer City Reservoir. Operational and emergency storage requirements consider serving demands in the Main-High, Upper, and Townsend Zones.

(f) The Main-Low/Old Highway Zones were evaluated assuming storage is available for use in the WTP Clearwell, Townsend Reservoir, Neer City Reservoir, and Old Highway Reservoir. Operational and emergency storage requirements consider serving demand systemwide.

4.3 WATER SYSTEM PERFORMANCE EVALUATION

The purpose of the water system performance evaluation is to identify necessary improvements to support the City's future water demands while meeting the City's recommended water system performance criteria. As discussed in Section 4.1, the City's projected water demand is a total system average day demand of approximately 528,000 gpd. This equates to a 10.5 percent increase in demand from 2020 to 2040.

The following evaluations were performed to assess distribution system performance under future water demand conditions:

- **Normal Operations – Peak Hour Demand – Existing Pipelines:** A peak hour flow condition was simulated for the existing distribution facilities to evaluate their capacity to meet the projected peak hour demand scenario. Peak hour demands are met by a combination of supply from storage reservoirs and pump stations.
- **Fire Flow Availability – Maximum Day Demand plus Fire Flow – Small Diameter Pipelines Replaced:** To evaluate the existing water system under the maximum day demand plus fire flow scenario, InfoWater's *"Available Fire Flow Analysis"* tool was used to determine the

available fire flow while meeting the maximum day demand plus fire flow performance criteria within the existing water system. Existing small diameter pipelines (4-inch diameter and smaller) were replaced with the minimum pipeline size of 8" for looped pipelines and 6" for dead-end pipelines, as established in *Chapter 3 Water System Performance Criteria*. Additional improvements required to meet fire flows, beyond small diameter main upgrades, were then identified under this evaluation.

Hydraulic analyses were conducted using a water system hydraulic model developed from the City's existing GIS. The performance criteria and results of the City's water distribution system evaluation are discussed below.

4.3.1 Normal Operations

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

4.3.1.1 Evaluation Criteria

The projected peak hour demand for the City was evaluated as 1,028 gpm (1.48 mgd). This represents peaking factors of 1.2 times the 2040 average day demand for industrial customer USG, and 4.2 times the 2040 average day demand for the remainder of the City. The City's peak hour demand minimum pressure performance criterion requires that 35 psi be maintained throughout the water system under peak hour conditions. In addition, new pipelines should be designed such that velocities do not exceed 5 ft/s.

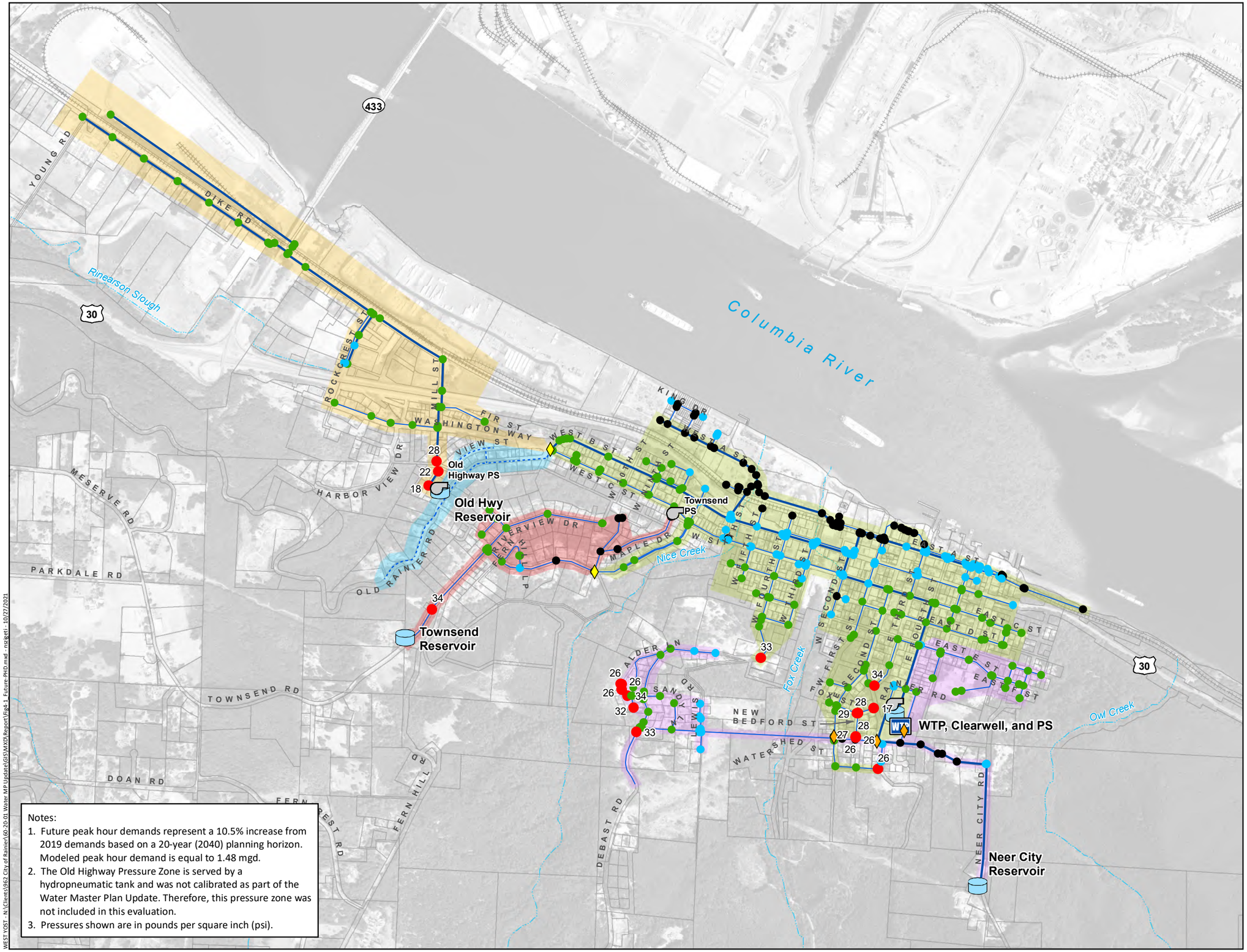
4.3.1.2 Evaluation Results

The City's existing water system is able to deliver peak hour demand while maintaining 35 psi at most service locations within the City. The model results illustrated in Figure 4-1 show that high elevation areas of the Main-High and Main-Low pressure zones fall below 35 psi.

To alleviate the low pressures in the Main-High Zone, located to the west of the WTP, it is recommended that the City serves this area from the Upper Zone. This can be achieved by closing the valve to the east of the tee at East Second Street and Fox Street and opening the valve to the north of the tee at New Bedford Street and East Norwood Street. Adjusting the isolation valve in this area will rezone the residences along Fox Street and New Bedford Street to the Upper Zone, served by the Neer City Reservoir. Service pressures will increase to over 120 psi, thus individual PRVs for customers in this area should be installed prior to rezoning.

In the Upper Zone along Debast Road, in the south-central portion of the system, low pressures are likely caused by a nearly 150 ft elevation increase between customers served along Lewis Road and customers served along Alder Lane, along the western-most edge of the Upper Zone. These pressures can be improved by raising the pressure setpoint of the PRV located at Rainier Road and East Norwood Street from 90 psi to 120 psi and installing individual PRVs for customers where static pressures exceed 80 psi.

Low pressures in the Main-Low Zone near the Old Highway Reservoir are due to an elevation difference of over 140 ft between the high elevation customers, located near the reservoir, and the low elevation customers, located along the Columbia River. Services located on the Old Highway Reservoir transmission pipeline where static pressures are below 35 psi should be equipped with individual booster pumps.



Notes:
1. Future peak hour demands represent a 10.5% increase from 2019 demands based on a 20-year (2040) planning horizon. Modeled peak hour demand is equal to 1.48 mgd.
2. The Old Highway Pressure Zone is served by a hydropneumatic tank and was not calibrated as part of the Water Master Plan Update. Therefore, this pressure zone was not included in this evaluation.
3. Pressures shown are in pounds per square inch (psi).

N

0 600 1,200

Scale in Feet

Symbology

- Storage Reservoir
- Pump Station
- Water Treatment Plant
- Pressure Regulating Valve
- Pressure Regulating/Sustaining Valve

System Pressure

- Less than 35 psi
- 35 - 80 psi
- 80 - 100 psi
- Greater than 100 psi

Existing Pipelines

- Less than 8-inches
- Greater than 8-inches
- Old Highway Hydropneumatic Zone

Pressure Zone

- Main-High
- Main-Low
- Old Highway
- Townsend
- Upper

WEST YOST

Figure 4-1

Future Peak Hour Demand Existing System Facilities

City of Rainier
Water Master Plan Update

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4.3.2 Fire Flow Availability

The maximum day demand plus fire flow scenario evaluates the fire flow availability in the City's water system under a maximum day demand condition, assuming all small diameter pipelines (4-inch diameter and smaller) have been replaced with the minimum pipeline size of 8-inch for looped pipelines and 6-inch for dead-end pipelines. Additional improvements were identified to meet the fire flow criteria outlined in Chapter 3. An overview of the evaluation criteria and a discussion of the results are presented below.

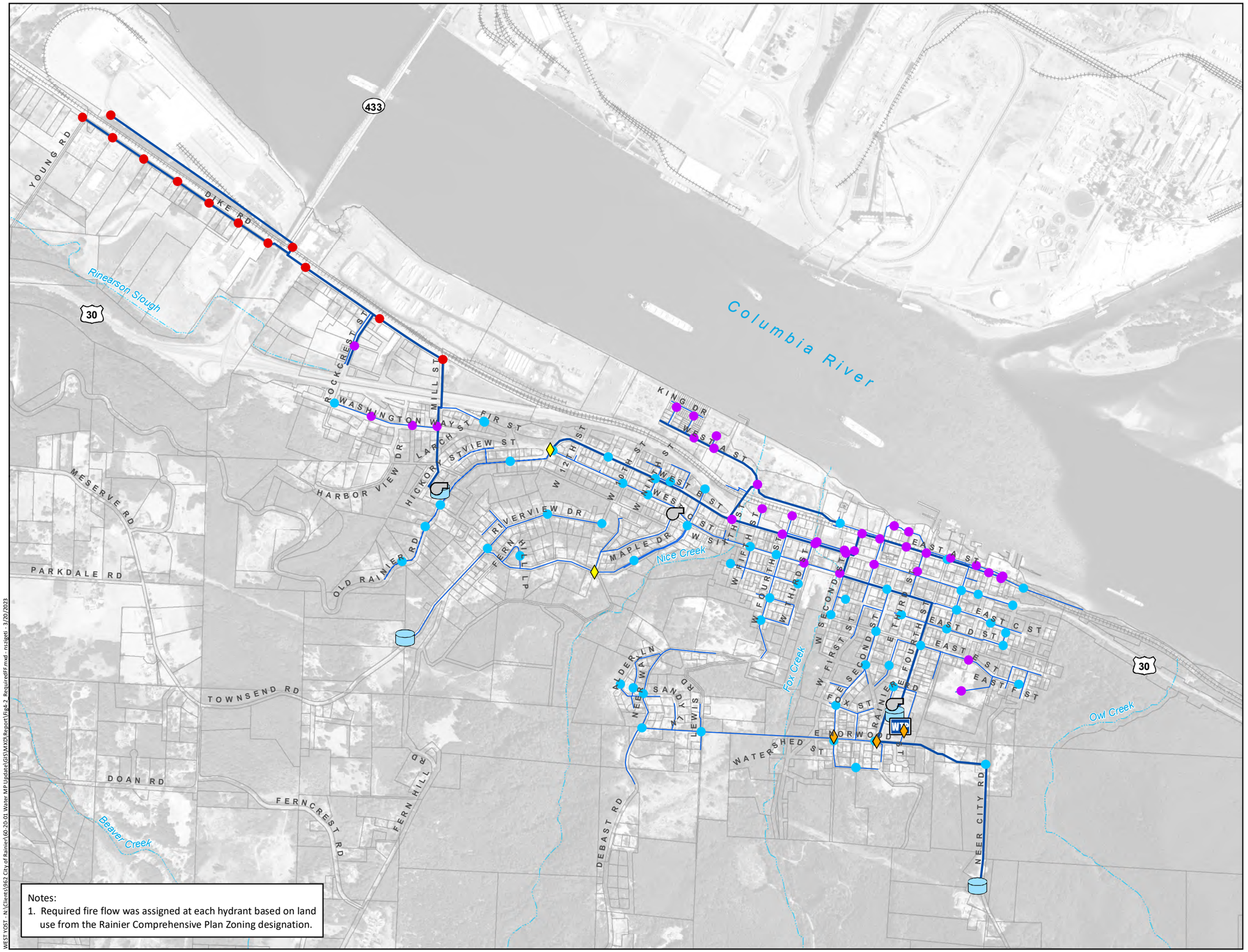
4.3.2.1 Evaluation Criteria

A projected maximum day demand of 734 gpm (1.09 mgd) for the City was used for the evaluation. This represents peaking factors of 1.2 times the 2040 average day demand for industrial customer USG, and 2.8 times the 2040 average day demand for the remainder of the City. The City's minimum pressure criterion requires that a 20 psi residual pressure be maintained throughout the water system under maximum day demand plus fire flow. In addition, new pipelines should be designed such that velocities not exceed 12 ft/s under fire conditions.

Fire flows were assigned to hydrant locations based on adjacent land use(s), per the City's 2015 Comprehensive Plan Zoning and fire flow requirements outlined in Chapter 3. Figure 4-2 shows the fire flow requirements assigned to hydrant locations. It should be noted that manual adjustments were made to some fire flow requirements to better represent the building purpose and size. For example, two hydrants adjacent to an existing women's in-patient addiction rehabilitation facility were assigned a fire flow of 3,000 gpm for 3 hours despite being zoned for Residential use. This building was previously a school and, due to its size, would require a fire flow greater than the residential requirement of 1,000 gpm.

4.3.2.2 Evaluation Results

Figure 4-3 shows the locations of deficient hydrants in the existing system under future maximum day demand conditions. Limiting pipeline improvements to strictly upsizing the City's small diameter mains will not be sufficient to meet fire flows in all locations while maintaining 20 psi residual pressure in the system. A majority of the City's commercial and industrial areas are deficient due to large fire flow requirements (3,000 gpm and greater). Additional improvements identified to improve fire flow availability are generally described as: (1) replacing 6-inch and 8-inch existing pipelines with larger 10-inch or 12-inch pipelines in high fire flow areas; and (2) looping existing dead-ends with new, short segments of new pipelines. The proposed supplemental improvements were targeted to small-diameter pipelines previously slated for replacement to minimize capital improvements, although some new construction is recommended. Zone-specific improvements are identified below and discussed in detail in Section 4.4. Figure 4-4 presents a comprehensive overview of the water system improvements recommended in this chapter, including the improvements discussed below.



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

0 600 1,200
Scale in Feet

Symbology

- Storage Reservoir
- Pump Station
- Water Treatment Plant
- Pressure Regulating Valve
- Pressure Regulating/Sustaining Valve

Existing Pipelines

- Less than 8-inches
- Greater than 8-inches
- Old Highway Hydropneumatic Zone

Fire Flow Requirement

- 1,000 gpm
- 3,000 gpm
- 4,000 gpm

WEST YOST

Figure 4-2

Required Fire Flow by Land Use

City of Rainier
Water Master Plan Update

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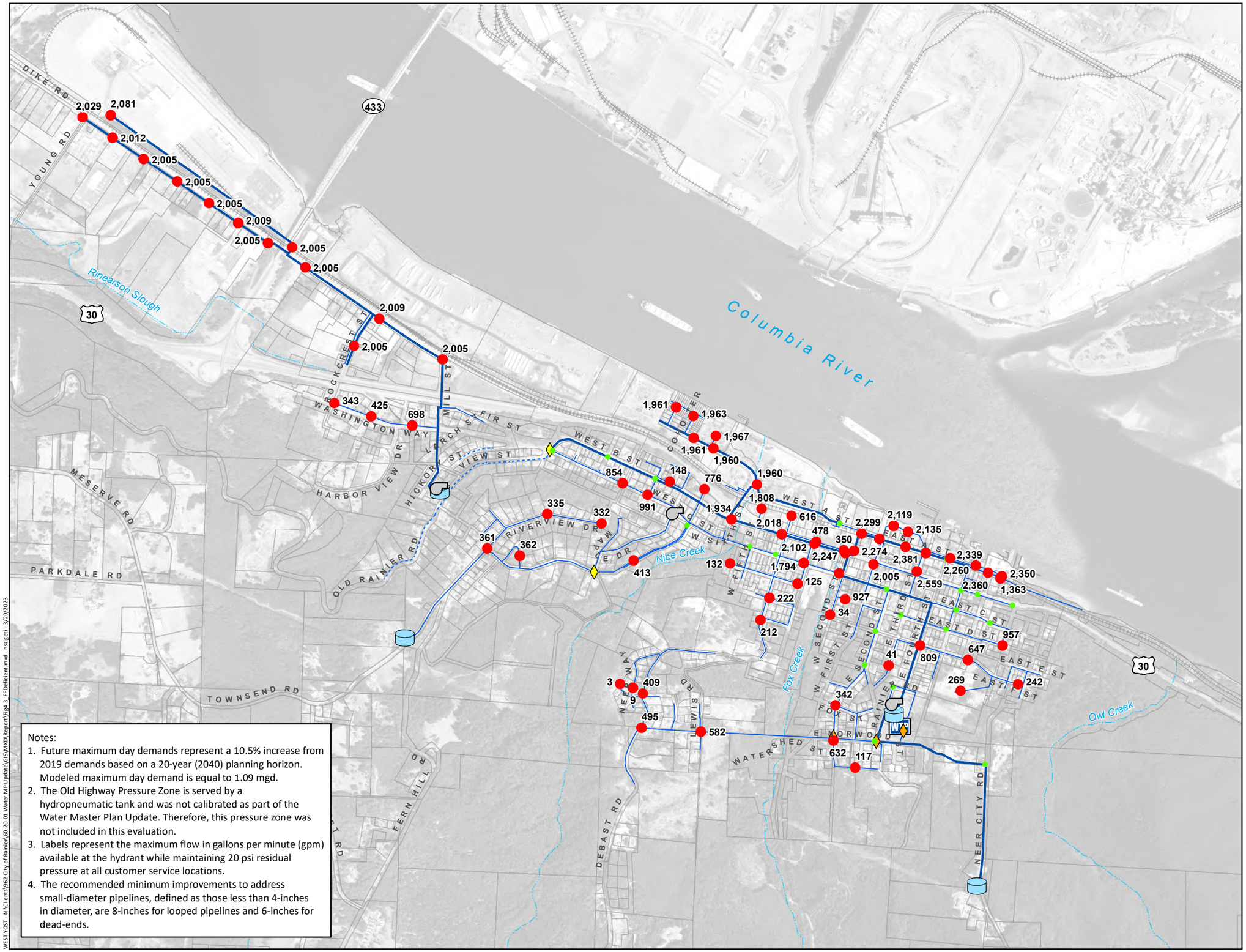

Notes:

1. Future maximum day demands represent a 10.5% increase from 2019 demands based on a 20-year (2040) planning horizon. Modeled maximum day demand is equal to 1.09 mgd.
2. The Old Highway Pressure Zone is served by a hydropneumatic tank and was not calibrated as part of the Water Master Plan Update. Therefore, this pressure zone was not included in this evaluation.
3. Labels represent the maximum flow in gallons per minute (gpm) available at the hydrant while maintaining 20 psi residual pressure at all customer service locations.
4. The recommended minimum improvements to address small-diameter pipelines, defined as those less than 4-inches in diameter, are 8-inches for looped pipelines and 6-inches for dead-ends.

N

0 600 1,200

Scale in Feet

Symbology

- Storage Reservoir
- Pump Station
- Water Treatment Plant
- Pressure Regulating Valve
- Pressure Regulating/Sustaining Valve

Existing Pipelines

- Less than 8-inches
- Greater than 8-inches
- Old Highway Hydropneumatic Zone

Available Fire Flow at Hydrant

- Meets Requirements
- Does Not Meet Requirements

WEST YOST

Figure 4-3

Future Fire Flow Availability Existing System Facilities

City of Rainier
Water Master Plan Update

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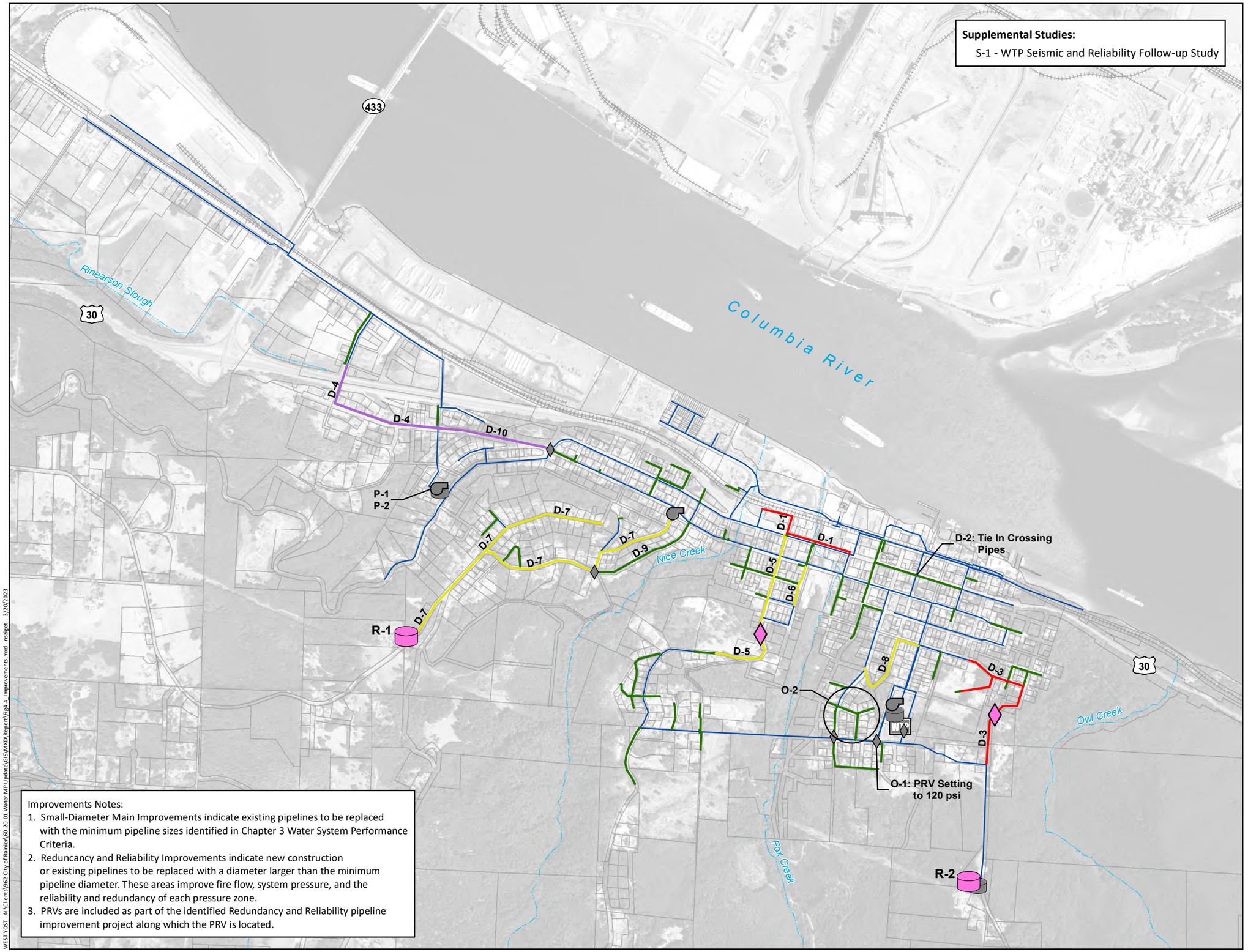

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WEST YOST

Figure 4-4

Proposed Water System Improvements

City of Rainier
Water Master Plan Update

Upper Zone: The Upper Zone requires significant improvements on the east and west edges of the zone. On the northwest side of the zone, a new 8-inch pipeline and PRV is recommended in Debast Road to provide redundancy and an additional connection to the Main-High Zone at W 4th Street (CIP Project D-5). To the east of the WTP, a new 10-inch pipeline and PRV are recommended from the tee at Neer City Road to E 10th Street and approximately 770 feet into East F Street to provide additional looping and improved capacity for fire flow (CIP Project D-3).

Townsend Zone: The Townsend Zone requires 8-inch minimum pipelines from the Townsend buried pump station to the Townsend Reservoir, and to the hydrants in the east of the zone (CIP Project D-7). This provides the minimum fire flow from the tank and pump station while maintaining 20 psi.

Main-High Zone: The Main-High Zone requires minor additional improvements beyond small-diameter main improvements. These improvements increase the available fire flow to the commercial areas of the City and are addressed by looping and pipeline replacement. Specific projects are described in detail in Section 4.4.4.

Main-Low Zone: Fire flow in the Main-Low Zone is primarily limited by the mile-long 12-inch dead-end pipeline that extends to the USG plant. However, as the available flow in these waterfront industrial areas is greater than 50-percent of the required flow (4,000 gpm), it is assumed that raw water from the Columbia River can be used in tandem with potable water to meet the requirement in a fire flow scenario. New 12-inch pipeline to create looping is proposed in Washington Way to Rockcrest Street (CIP Project D-4) to improve fire flow availability in the industrial waterfront and provide additional redundancy.

4.4 SUMMARY OF FINDINGS AND RECOMMENDATIONS

The recommended improvements proposed to eliminate the water system capacity and performance deficiencies identified in the preceding evaluations are summarized below. These recommendations only identify improvements at a master planning level and do not constitute a design of such improvements. Subsequent detailed design will be required to determine the exact sizes and/or locations of these proposed improvements. The estimated costs for these recommended improvements are discussed in Chapter 6.

Figure 4-4 illustrates the improvements recommended to the City's water system to meet the City's performance criteria. Improvements are divided into two categories defined below:

- **Small Diameter Mains Improvements**: Existing small-diameter mains that should be replaced with the minimum pipeline diameter recommended in Chapter 3.
- **Redundancy and Reliability Improvement**s: Proposed improvements to meet the performance criteria described in Chapter 3. These improvements include the construction of new pipelines and the replacement of existing pipelines.

The redundancy and reliability improvements shown in Figure 4-4 are described in detail below.

4.4.1 Supply and Water Treatment

- **S-1 – WTP Seismic and Reliability Follow-up Study:** As described in Chapter 5 Seismic Risk and Mitigation Plan, a geotechnical, structural, mechanical, and seismic investigation of the WTP, reservoirs, and pump stations is recommended. This study should also determine what improvements are needed to properly treat Fox Creek water, as described in this chapter. Discussions with City staff indicate the following items, at a minimum, should be included as part of this study:
  - Existing Water Treatment Plant
  - Replace/upgrade chlorine and coagulant chemical feed pumping systems, chemical storage, and containment
  - Upgrade and replace the SCADA and Programmable Logic Controller (PLC) to add controls for remote chemical adjustments
  - Refurbish filter #3 (filter #1 was refurbished in 2021/2022 and filter #2 was refurbished in 2019/2020)
  - Refurbish/replace the filter blowers (3) and mechanical valves
  - Add level control for effluent valve control to all three filters (currently equipped with air pressure controls)
  - Replace turbidimeter for Columbia River raw water
  - Add turbidimeters on all clarifiers (3)
  - Evaluate existing instrumentation that is unused
  - Refurbish the backwash lagoon and cracks in the concrete, replace drain lines, and evaluate alternative options for pumping to sewer
  - Improvements to Treat Fox Creek Water
  - Add new chemical feed system for pH adjustment, including chemical storage and containment
  - Add pH influent monitoring to dose chemicals for pH control
  - Repair/replace flow control valves and flow meters for the Columbia River and Fox Creek raw water pipelines
  - Replace Fox Creek turbidimeter
  - Evaluate alternatives for blending water sources prior to treatment
  - Conduct a biweekly sampling and analysis program for 1 year
  - Evaluate full replacement of Fox Creek raw water pipeline (partially replaced in 2012)

4.4.2 Pumping

- **P-1 – Old Highway Fire Pump:** Replace the existing (unknown capacity) fire pump with a pump that can serve the minimum required fire flow in the Old Highway Zone (1,000 gpm).
- **P-2 – Old Highway Hydropneumatic Tank:** Replace the existing problematic hydropneumatic tank at the Old Highway Pump Station to prevent wear on the Old Highway Pump Station improve supply reliability to the Old Highway Zone.

4.4.3 Storage Reservoirs

- **R-1 – New Townsend Reservoir (0.08 MG):** A second Townsend Reservoir of at least 80,000 gallons should be constructed next to the existing tank to make up the storage deficit. A retired redwood storage tank exists at the site and could be demolished to provide space for another storage tank.
- **R-2 – New Neer City Reservoir (0.13 MG):** A second Neer City Reservoir of at least 130,000 gallons should be constructed at the same elevation as the existing Neer City Reservoir to make up the storage deficit.

4.4.4 Pipelines

- **D-1 – W B Street/W A Street – New 10-inch Pipeline:** Replace the existing 4-inch pipeline along B Street from W Second Street to W Fourth Street and construct approximately 400 feet of new 10-inch pipeline along A Street (south of the rail-line) between W Fourth Street and W Fifth St. The result is a continuous 10-inch main which would loop into the existing system at the intersection of W Fifth Street and A Street to the west, and W Second Street and B Street to the east. This eliminates long dead-end pipes and improves fire flow.
- **D-2 – Intersection of E B St and E Third Street – Looping:** When replacing the existing 4-inch small-diameter main along E B St, the existing crossing 6-inch pipeline along E Third Street should be tied into the new pipeline. This will provide looping and improve fire flow.
- **D-3 – Neer City Road/E F Street – New 10-inch Pipeline and PRV:** Construct approximately 1,110 ft of new 10-inch pipeline from the existing tee on Neer City Road, north along Neer City Road to the existing system on E Tenth Street. A PRV must be placed along this length of pipeline to reduce pressures from the Neer City Reservoir. In addition, replace existing small-diameter mains with 10-inch pipeline to create a continuous 10-inch main from the tee to the hydrants surrounding the former high school building. This provides looping to this area and allows the Neer City Reservoir to deliver the 3,000 gpm fire flow requirement.
- **D-4 – Washington Way/Rock Crest Street – New 12-inch Pipeline:** Replace the existing 4-inch pipeline along Washington Way from Mill Street to Rock Crest Street with a 12-inch pipeline. In addition, construct approximately 500 ft of new 12-inch pipeline to connect the dead-ends on either side of the Columbia River Hwy. This provides redundancy and looping and improves fire flow to the commercial and industrial areas on either side of the highway.
- **D-5 – Debast Road/W Fourth Street – New 8-inch Pipeline and PRV:** Replace the existing pipelines along W Fourth Street from W C Street to the dead-end south of W F Street. In addition, construct new 8-inch pipeline to continue south and connect to the dead-end along Debast Road. A PRV should be included along this pipeline to maintain the zone boundary between the Main-High Zone and Upper Zone. This will provide looping and redundancy to the area.
- **D-6 – W Third Street – New 8-inch Pipeline:** Replace the existing small diameter mains and construct new 8-inch pipeline along W Third Street between W C Street and W E Street. This loops several dead ends and improves fire flow.

- **D-7 – Maple Drive/Fern Hill Road/Riverview Drive/Townsend Road – New 8-inch Pipeline:** Replace all existing mains between the Townsend buried pump station and the Townsend reservoir with 8-inch pipeline, as well as the entire pipeline along Riverview Drive. This greatly improves fire flow in the Townsend Zone.
- **D-8 – E Third Street/E E Street – New 8-inch Pipeline:** When replacing the small-diameter pipelines south of E E Street along E Third Street, construct a new 8-inch pipeline connecting the dead-ends with the pipeline along E Second Street. This provides looping to the area and improves low pressures.
- **D-9 – Fern Hill Road – Relocate Services:** When replacing the existing 4-inch small-diameter main along Fern Hill Road from the PRV to West Sixth Street, relocate any existing customer services from parallel pipes to the newly constructed pipe. This eliminates dead-ends and improves customer service pressures. Costs associated with this improvement are accounted for under the small-diameter mains improvements budget.
- **D-10 – Washington Way – New 12-inch Pipeline:** Construct approximately 1,500 feet of new 12-inch pipeline from the existing control valve in Washington Way and West B Street to the existing tee at Washington Way and Mill Street. This provides redundancy and looping to the Main-Low Zone.

4.4.5 Operational

- **O-1 – Rainier and Norwood PRV – Change Setting:** Raise the setting of the PRV located at Rainier Road and E Norwood Street to 120 psi. This provides sufficient pressure to the high-elevation services across Fox Creek to the west to meet the City's minimum pressure requirement of 35 psi. However, this adjustment will cause pressures in excess of 130 psi in the 8-inch pipeline along Norwood Street, and any service or distribution laterals along this pipeline should be equipped with individual PRVs.
- **O-2 – Adjust Upper/Main-High Zone Isolation – Close Valve at Fox Street and E Second Street:** If possible, services adjacent to the 8-inch pipeline in E Norwood Street and at a similar elevation to the WTP should be served by the Upper Zone to take advantage of the high pressures provided by the Neer City Reservoir. Otherwise, they are served by gravity from the WTP and experience pressures below the minimum pressure requirement during peak hour demand of 35 psi.

CHAPTER 5
Seismic Risk and Mitigation Plan

5.1 SEISMIC RISK AND MITIGATION PLAN

This chapter discusses the seismic hazards within the City's water service area, the potential impacts to the water system during a seismic event, and a list of recommended projects or investigations.

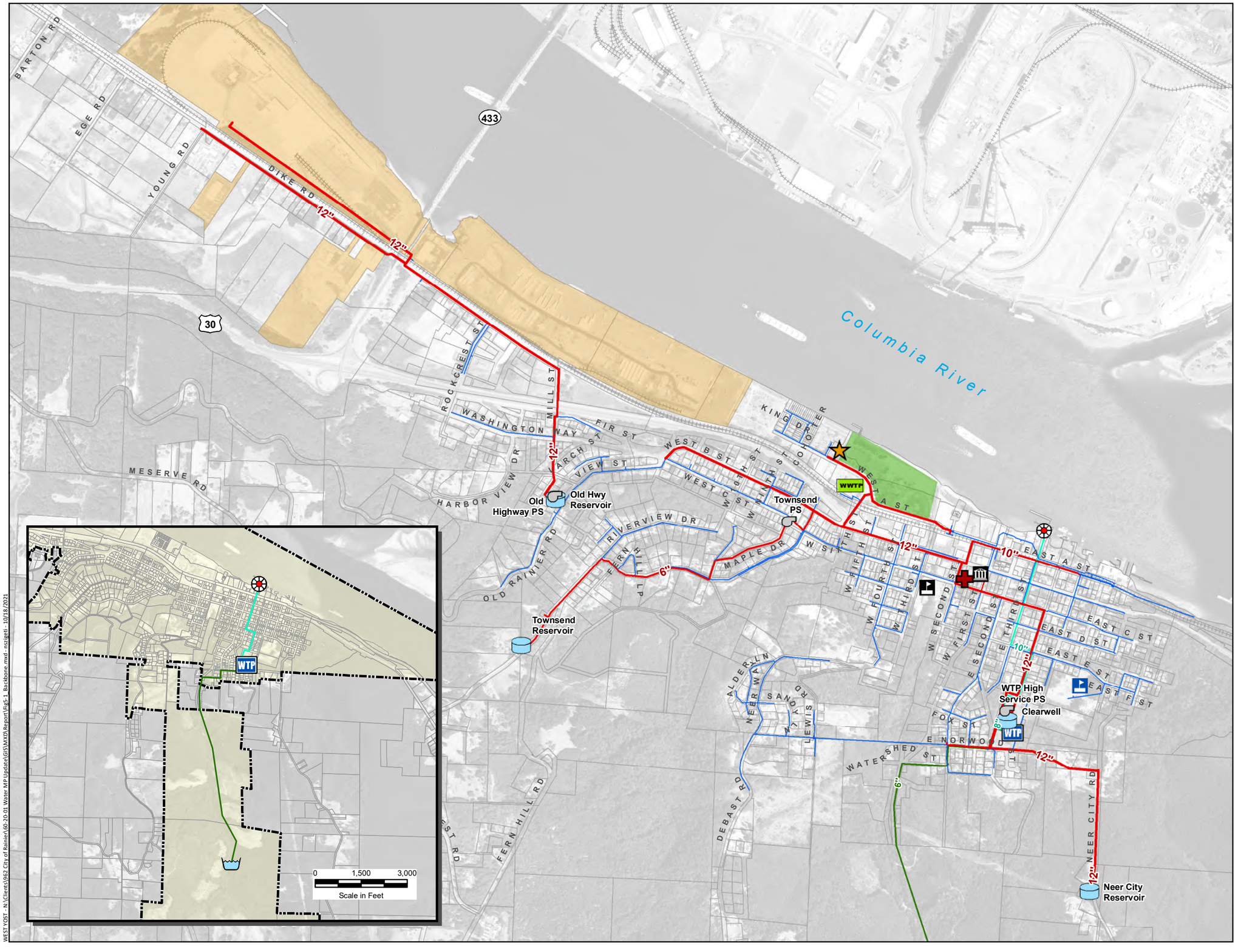
The City of Rainier is located in an area of moderate to moderate/heavy damage potential following a Magnitude 9.0 Cascadia Subduction Zone (CSZ) earthquake according to the DOGAMI seismic hazard maps. Due to its potential risk, a seismic risk assessment and mitigation plan for the City's water system shall be developed in accordance with the OHA requirements and the Oregon Resilience Plan (ORP) goals. The purpose of this mitigation plan is to ensure reasonable levels of emergency water supply and to help planning the improvement of the resiliency of the City's critical water system backbone. The following sections describe the key components of this plan:

- Water System Backbone with identification of essential water facilities, critical customers, and location of emergency water distribution
- Seismic Hazards Evaluation of the critical water facilities and backbone pipe vulnerability assessment
- Seismic Resiliency Results for Critical Facilities
- Pipeline Fragility Results

5.2 WATER SYSTEM BACKBONE

A water system "backbone" is the infrastructure required to maintain adequate supply to essential facilities and critical customers in the City for post-earthquake response, public health and safety. Backbone infrastructure and water facilities have been identified for the City water system and include the Columbia River intake, the raw water pipeline, the Water Treatment Plant, the tanks and reservoirs in the City, and the critical pipelines. A map of the backbone system for the City of Rainier is shown on Figure 5-1.

Following a seismic event, water supply will be disrupted and many of the residential and industrial water services will be damaged. It is important to identify critical customers and water supply locations to serve the community after such an event. Alternative sources of distribution may include the use of an emergency point of distribution (POD) trailer to supply drinking water to customers, the distribution of bottled water or the stored water in the City's tanks and reservoirs if unaffected by the seismic event. Locations for water distribution include the Rainier City Hall, police department, fire station, Public Works building located at the Wastewater Treatment Plant, City Park, the Rainier Elementary and an Inpatient Addiction facility (locations shown on Figure 5-1).



Symbology

- Pump Station
- Water Treatment Plant
- Fox Creek Reservoir
- Storage Reservoir
- City Limit

Potable Water Pipelines

- Non-Backbone Pipeline
- Backbone Pipeline

Raw Water Pipelines

- Columbia River
- Fox Creek
- Columbia River Intake

Critical Locations

- Columbia River Fire and Rescue
- Rainier City Hall
- Rainier Elementary School
- Rehabilitation Center
- Rainier Senior Center
- Wastewater Treatment Plant
- City Park
- Industrial Customers

WEST YOST

Figure 5-1

Backbone Identification Map

City of Rainier
Water Master Plan Update

WEST YOST - N:\Clients\962 City of Rainier\60-20-01 Water MP Update\GIS\MXD\Report\Fig5-1_Backbone.mxd - rossjgett - 10/18/2021

5.3 SEISMIC HAZARDS EVALUATION

To help the City prepare and appropriately invest in resilience planning for its water system backbone, a geotechnical and structural seismic hazards evaluations were developed. As previously mentioned, a 9.0 CSZ earthquake was selected for the earthquake hazards analysis. This section includes the methodology used to evaluate the seismic hazards and pipeline fragility within the City's water system backbone.

5.3.1 Geotechnical Seismic Risks and Hazard Mapping

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

The City of Rainier lies along the southwest bank of the Columbia River and at the eastern foot of the northern Coast Range. The geologic units under the City include volcanic rocks, terrestrial sedimentary rocks, and alluvial sediments.

The analysis of the seismic hazards in the City's service area is based on geological information, geotechnical explorations, historic well logs, background data, and available earthquake scenario maps (DOGAMI maps). Seismic hazards to be estimated include strong ground shaking (peak ground velocity and accelerations), liquefaction settlement, lateral spreading displacement, and seismic-induced landslides.

An additional site-specific investigation was conducted by Delve Underground to further identify and estimate the intensity of the seismic hazards at the Columbia River Intake facility and at the Water Treatment Plant site.

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

5.3.2 Structural Seismic Resiliency Evaluation

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

5.3.3 Pipeline Fragility Evaluation

To estimate the likelihood of damage to buried pipe in a seismic event, the American Lifelines Alliance (ALA) developed methods published in the report *Seismic Fragility Formulations for Water Systems* (ALA 2001) for estimating seismic fragility for water pipes. These methods are based on the frequency of pipe

breaks in past earthquakes and correlating this with the ground shaking and measured ground movements (from liquefaction and landslides) at the site of the break. A break is defined as pipe damage severe enough to require a repair. Water agencies frequently use these methods to estimate the seismic resiliency of their water system backbone pipes.

The ALA guideline recommends using two pipe vulnerability functions as shown in Table 5-1 to evaluate the repair rates (RR) for a large inventory of pipelines such as a water distribution system. The first function estimates a repair rate (RR) per 1,000 LF of pipe due to seismic wave propagation (ground shaking), and the second function estimates a repair rate (RR) per 1,000 LF of pipe due to permanent ground deformation (liquefaction, lateral spreading, and seismic landslides).

Table 5-1. Buried Pipe Vulnerability Functions

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

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

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

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

The findings of the City pipeline fragility evaluation are summarized in the geotechnical seismic hazards evaluation report in Appendix A.

5.4 SEISMIC RESILIENCY RESULTS FOR CRITICAL FACILITIES

As shown in Figure 5-1, the City's critical water facilities include the Columbia River raw water intake, the water treatment plant, the Neer City reservoir, the Old Highway reservoir, the Townsend reservoir, and the Fox Creek reservoir, dam, and raw water pipeline.

For a magnitude 9 CSZ event, ground shaking is estimated to range from 7 to 11 inches per second on consolidated soils and up to 21 inches per second in areas with recent alluvial deposit (Columbia River bank and along small drainages). This will cause moderate to heavy potential damage depending on the location of the site. Liquefaction settlement and lateral spreading are estimated to be low in the City except along the Columbia River and smaller drainage areas where liquefaction settlement (up to 12 inches) and lateral spreading (up to 15 feet) are high. Seismic-induced landslides are relatively low in the City except along the Fox Creek raw water pipeline and near the Fox Creek reservoir dam where numerous landslides have occurred.

The results of the analyses for the City's pipeline backbone indicate that the water system extends across a wide range of seismic hazards with the major hazard concerns being high liquefaction and lateral spreading near the Columbia River and in some localized drainage areas. The structural evaluation indicate that the majority of the critical water facilities are in reasonable structural condition.

These findings were applied to the critical water facilities to assess their vulnerability during a 9.0 CSZ seismic event and are summarized below. Additional details regarding the analyses of these facilities are provided in Appendices A and B.

5.4.1 Columbia River Raw Water Intake

The Columbia River raw water intake facility is located on the south bank of the Columbia River and consists of a pump house and an intake pipe that extends approximately 150 feet offshore and protected by a group of timber pilings. The pump house is a 15 by 15 feet, single-story, reinforced CMU block building that sits on an 8-foot diameter, 32-foot-deep concrete wet well. The intake was built in 1968 and retrofitted in 2000 when the pumps and housing structure were replaced.

5.4.1.1 Seismic Hazards

A site-specific geotechnical investigation was conducted at the Columbia River Intake by Delve Underground. One exploratory boring was drilled at the site. Test results of the soil sample indicate that the subsurface conditions consist of artificial fill (poorly graded sand likely from dredged river materials), over loose silty sand Holocene alluvium, and marine sedimentary and volcanic rocks. Groundwater level was observed at 13 feet below ground surface (approximately water surface level of the Columbia River).

Based on the site-specific evaluation, findings are as follows:

- 0.35g peak ground acceleration (PGA)
- 20 in/sec peak ground velocity (PGV)
- Around 8 inches of vertical, post-liquefaction settlement
- 5-11 feet lateral spread displacements
- Earthquake-induced landslide is estimated to be high

The results show that the Columbia River Intake facility is at high risk for significant damage to possibly a complete loss of serviceability after a 9.0 CSZ earthquake. To mitigate the liquefaction and lateral spreading hazards at the intake facility, extensive ground improvement (such as jet grouting), upgrades at the pump station, and mitigation of the in-water intake pipe and screen will be required. This solution would be very extensive and expensive with potential environmental risks due to grout leakage into the river. The City expressed some concerns with this option and is not interested in pursuing this solution. A couple of other alternatives could be considered:

- Rebuild the raw water intake facility on a site not susceptible to liquefaction and lateral spreading hazards.
- Utilize the Fox Creek Reservoir as an alternative raw water source while repairs are being performed on the existing Columbia River intake (assuming the existing facility could be repaired).

A suggested location for a new raw water intake facility is an area on the east side of the City along the Columbia River where shallow bedrock are anticipated. The new pump station building is estimated to be approximately 20 feet by 40 feet reinforced CMU block building housing three pumps. The raw water would be pumped through approximately 1,500 feet of 10-inch diameter ductile iron pipe along E C Street installed by a combination of trenchless technology and open cut. The new pipe would then be connected to the existing pipe on E Third Street. The proposed in-water intake pipe would be approximately 300 feet, undercrossing the railroad tracks, with a screen at the entrance and an aeration system to clean the screen; the intake pipe is anticipated to be installed by trenchless technology. A preliminary layout is included in Appendix C. Preliminary cost for the construction of a new raw water intake facility is estimated to be around $7.5 million including design, land acquisition, permitting, and construction (see Appendix C for a cost breakdown).

Evaluation and additional information on the Fox Creek dam and reservoir are included in Section 5.3.6.

5.4.1.2 Structural Deficiencies

The following items summarize the structural evaluation findings:

- **Wall Anchorage:** The roof diaphragm is not connected to the CMU block walls to adequately resist out of plane forces on the gable sides of the structure.
- **Transfer to Shear Walls:** The roof diaphragm does not appear to be directly connected to the CMU block walls on the gable sides of the structure.
- **Deep Foundations:** The structure is placed on top of a precast concrete manhole. The manhole is placed in sections. The sections have been bolted together, but it is unclear if the manhole is capable of transferring lateral forces from the structure to the soil.
- **Sloping Sites:** The site slopes dramatically down to the river. The geotechnical investigation at that location concluded that the pump house is susceptible to lateral spreading.
- The downspout on the back side of the structure should be routed downhill away from the foundation.
- The bottom of the concrete slab foundation is exposed on nearly all sides. Minimum frost depth and lateral resistance is typically 12 inches.

5.4.2 Water Treatment Plant

The Water Treatment Plant was constructed new in two phases in 2000 and 2001, in the hills south of the City. Facilities at the site include a 50 by 60 feet main treatment building, a 0.5 MG storage clearwell, a 0.25 MG backwash lagoon basin, an approximately 45 by 50 feet storage building (not included in the structural evaluation). The main treatment building consists of CMU block walls with a wood framed mezzanine, a generator room and a loading dock. The 0.5 MG tank is a welded steel tank on a concrete foundation with no anchor bolts. The backwash lagoon is a concrete structure located just west of the main building.

5.4.2.1 Seismic Hazards

A site-specific geotechnical investigation was conducted at the Water Treatment Plant by Delve Underground. Two exploratory borings were drilled at the site. Test results of the soil samples indicate that the subsurface conditions consist of colluvium, alluvial terrace deposits, and volcaniclastic sedimentary rocks. Groundwater level was not observed during the exploratory borings at the site.

Based on the site-specific evaluation, findings are as follows:

- 0.22g peak ground acceleration (PGA)
- 10 in/sec peak ground velocity (PGV)
- Low risk of liquefaction settlement, lateral spreading displacement, and earthquake-induced landslide

Results show that the Water Treatment Plant site has a low risk of seismic hazards and is considered to be seismically resilient from a geotechnical and geologic hazards perspective. A structural evaluation for strong ground shaking has also been performed at the Water Treatment Plant site to identify any potential vulnerabilities.

5.4.2.2 Structural Deficiencies

The following items summarize the structural evaluation findings:

A. Water Treatment Plant - Building

- **Transfer to Shear Walls:** The roof diaphragm does not appear to be directly connected to the CMU block walls on the gable sides of the structure.
- **Suspended Contents:** Piping and vacuum lines are suspended from the structure and are free to swing or move but may damage themselves or adjoining components.
- **Tall Narrow Equipment:** There are several pieces of equipment, like the compressor, more than 6 feet tall that should be anchored to the floor or adjacent walls.
- **Conduit Couplings:** Conduit greater than 2.5 inches should have flexible couplings.
- The roof gutters appear to leak at the joints causing the fascia to rot. If left unmaintained the structure will be damaged and need to be replaced.
- The downspouts are terminated very near the base of the walls. The downspouts should be routed away from the structure to prevent erosion and fungus build up on the structure.
- Efflorescence was observed on the west side of the structure in the concrete wall near the stairs to the loading dock. This typically occurs when moisture is transmitting through the concrete wall. It may be possible that water is leaking inside the building and migrating to the west. If moisture continues to transmit through the wall structural damage will result. It is recommended that the trench & piping inside the Filter Building be checked for leaks and water inside the building routed to reliable outlets.
- The rust and corrosion around the base of the steel pipe supports should be treated, repaired and properly coated to prevent further corrosion.
- The sodium hypochlorite tank should have clips bolted to the concrete slab to prevent lateral movement and overturning during a seismic event.
- The caustic soda tank should have clips bolted to the concrete slab to prevent lateral movement and overturning during a seismic event.

B. Water Treatment Plant – Clearwell

- It is recommended that the steel tank be bolted to the concrete foundation to prevent sliding and overturning during a seismic event.
- Clean and remove any moss, mold, rust and corrosion. Then touch up paint any areas to prevent loss of steel thickness.

C. Water Treatment Plant – Backwash Lagoon

- No deficiencies

5.4.3 Neer City Reservoir

The Neer City Reservoir was constructed in 2000 in the hills to the southeast of the City. This reservoir is a 0.5 MG welded steel tank on a concrete foundation with no anchor bolts.

5.4.3.1 Seismic Hazards

Based on the geotechnical evaluation report, findings are as follows:

- 7-11 in/sec peak ground velocity (PGV)
- Low risk of liquefaction settlement, lateral spreading displacement, and earthquake-induced landslide

Results show that the Neer City Reservoir site has a low risk of seismic ground deformation.

5.4.3.2 Structural Deficiencies

The following items summarize the structural evaluation findings:

- It is recommended that the steel tank be bolted to the concrete foundation to prevent sliding and overturning during a seismic event.
- Clean and remove any moss, mold, rust and corrosion. Then touch up paint any areas to prevent loss of steel thickness.

5.4.4 Old Columbia River Highway Reservoir

The Old Columbia River Highway Reservoir was constructed in 1957 in the hills to the west of downtown. This reservoir is a 0.75 MG welded steel tank on a concrete foundation with no anchor bolts.

5.4.4.1 Seismic Hazards

Based on the geotechnical evaluation report, findings are as follows:

- 7-11 in/sec peak ground velocity (PGV)
- Low risk of liquefaction settlement, lateral spreading displacement, and earthquake-induced landslide

Results show that the Old Columbia River Highway Reservoir site has a low risk of seismic ground deformation. However, a significant seepage from the slope to the east above the tank was observed. The existing French drain is no longer functional. New drainage improvements should be implemented to reduce the risk of slope instability above the reservoir; if not mitigated, this issue may have the potential in long-term to trigger a large-scale landslide and impact the reservoir.

5.4.4.2 Structural Deficiencies

The following items summarize the structural evaluation findings:

- It is recommended that the steel tank be bolted to the concrete foundation to prevent sliding and overturning during a seismic event.
- Clean and remove any moss, mold, rust and corrosion. Then touch up paint any areas to prevent loss of steel thickness.

5.4.5 Townsend Road Reservoir

The Townsend Road Reservoir was constructed in 2099 in the hills to the southwest of the City. This reservoir is a bolted welded steel tank on a concrete foundation with a capacity of 15,000 gallons.

5.4.5.1 Seismic Hazards

Based on the geotechnical evaluation report, findings are as follows:

- 7-11 in/sec peak ground velocity (PGV)
- Low risk of liquefaction settlement, lateral spreading displacement, and earthquake-induced landslide

Results show that the Townsend Road Reservoir site has a low risk of seismic ground deformation.

5.4.5.2 Structural Deficiencies

The following items summarize the structural evaluation findings:

- It is recommended that the steel tank be bolted to the concrete foundation to prevent sliding during a seismic event.
- Remove the rust and corrosion then clean and paint the tank to prevent corrosion and loss of steel thickness.

5.4.6 Fox Creek Reservoir, Dam, and Raw Water Pipeline

The Fox Creek reservoir is located in the hills to the southeast of the City; it is an old facility built in the early 1900s and surrounded by steep slopes nearby. The existing dam is a wooden structure supported on concrete grade-beam foundation. The dam is approximately 25 feet tall, 50 feet wide and 200 feet long at the crest. The volume of the reservoir is approximately 1,400-acre feet. The dam is classified as high hazard due to the proximity of downstream residences. Review of available geological maps indicates that the dam appears to be founded on marine sedimentary rocks and the abutments appear to be stable based on a site visit with City staff on December 5, 2022. The presence of landslide debris around the

reservoir has also been previously documented. Seismic settlement, lateral spreading and landslides may all be a hazard at this site.

The Fox Creek raw water pipeline was recently reconstructed mainly on the west side of the creek under the access road to the reservoir. The road passes through a steep banked canyon with several mapped landslide hazards. The seismic landslide hazard is considered to be high in this area.

The City mentioned an interest in using Fox Creek as a back-up source of water in the event the Columbia River intake facility fails after a seismic event. Preliminary cost to improve/rehabilitate the existing dam is estimated to be around $6 million including design, permitting, and construction (see Appendix A). However, additional studies including geotechnical and stability analyses, hydraulic and dam break analyses are recommended. Based on the results of these analyses, rehabilitation of the existing dam may not be feasible, and rebuilding the entire dam might be necessary which would be more expensive.

Structural seismic evaluation of the Fox Creek dam and raw water pipeline has not been performed for this Water Master Plan.

5.4.7 General Non-Structural Considerations

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

- **Fire Suppression Piping:** Make sure piping is anchored and braced in accordance with current NFPA standards. Consider anchoring and bracing all piping in all facilities.
- **Hazardous Material Storage:** Some chemicals used in the treatment process or used during regular cleaning and maintenance processes may be considered hazardous when spilled. Items storing these chemicals should be restrained to prevent displacement, tipping, or falling.
- **Hazardous Material Distribution:** Natural gas piping should be anchored or braced adequately to prevent damage that might allow the hazardous material to release.
- **Shutoff Valves:** Piping containing hazardous material, including natural gas, should have shutoff valves or other devices to prevent spills or leaks.
- **Flexible Couplings:** Hazardous material, ductwork, and piping, including natural gas piping, should have flexible couplings.
- **Light Fixtures Lense Covers:** Make sure lens covers on light fixtures are attached with safety devices and add safety devices if necessary.
- **Industrial Storage Racks**: Industrial storage racks or similar items that are more than 12 feet high should be anchored to the floor.
- **Tall Narrow Cabinets**: Cabinets, lockers, bookshelves, etc. more than 6 feet high and with height-to-depth ratios exceeding 3:1 should be anchored to the floor or wall.
- **Fall-Prone Contents:** Equipment, stored items weighing more than 20 pounds and more than 4 feet above the floor should be braced or restrained.
- **Fall-Prone Equipment:** Equipment weighing more than 20 pounds and more than 4 feet above the floor should be braced or restrained.

- **In-Line Equipment:** Equipment installed in line with a duct or piping system, with an operating weight more than 75 pounds should be laterally braced independent of the duct or piping system.
- **Tall Narrow Equipment:** Equipment, tanks, etc. more than 6 feet high and with height-to-depth ratios exceeding 3:1 should be anchored to the floor or wall.
- **Suspended Equipment:** Equipment suspended without lateral bracing should be free to swing or move with the structure without damaging itself or adjoining components.
- **Heavy Equipment:** Floor supported or platform supported equipment weighing more than 400 pounds should be anchored to the structure.
- **Conduit Couplings:** Conduit greater than 2.5 inches should have flexible couplings.
- **Flexible Couplings:** Fluid and gas piping should have flexible couplings.
- **Fluid And Gas Piping:** Fluid and gas piping should be anchored and braced to the structure to limit spills or leaks.

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

5.5 PIPELINE FRAGILITY RESULTS

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

The City backbone pipeline system is composed of several different materials. The vast majority of pipes is made of cast iron (over 90 percent of the system), followed by some galvanized steel (around 5 percent), ductile iron, steel, and some unknown materials. The ALA methods were applied to these backbone pipelines and the Columbia River raw water pipeline to assess its vulnerability during a seismic event. Table 5-2 summarizes the estimated breaks in the backbone system and Table 5-3 summarizes the estimated breaks in the Columbia raw water pipeline.

Table 5-2. Estimated Breaks Due to Shaking by Pipeline Material in Backbone

Material Description	Symbol	Maximum Diameter, in	Pipeline Length, ft	Estimated Breaks by PGV	Estimated Breaks by PGD
Cast Iron	CI	12	33,179	1.0	90.3
Ductile Iron	DI	12	49	0.0	0.1
Steel Small	GALV	12	1,504	0.0	2.1
Steel Large	STL	16	608	0.0	2.5
Unknown	UNK	12	291	0.0	0.0
		Total	**35,624**	**1.0**	**95.0**

Note: Estimated breaks are back calculated from RR per pipeline segment length.

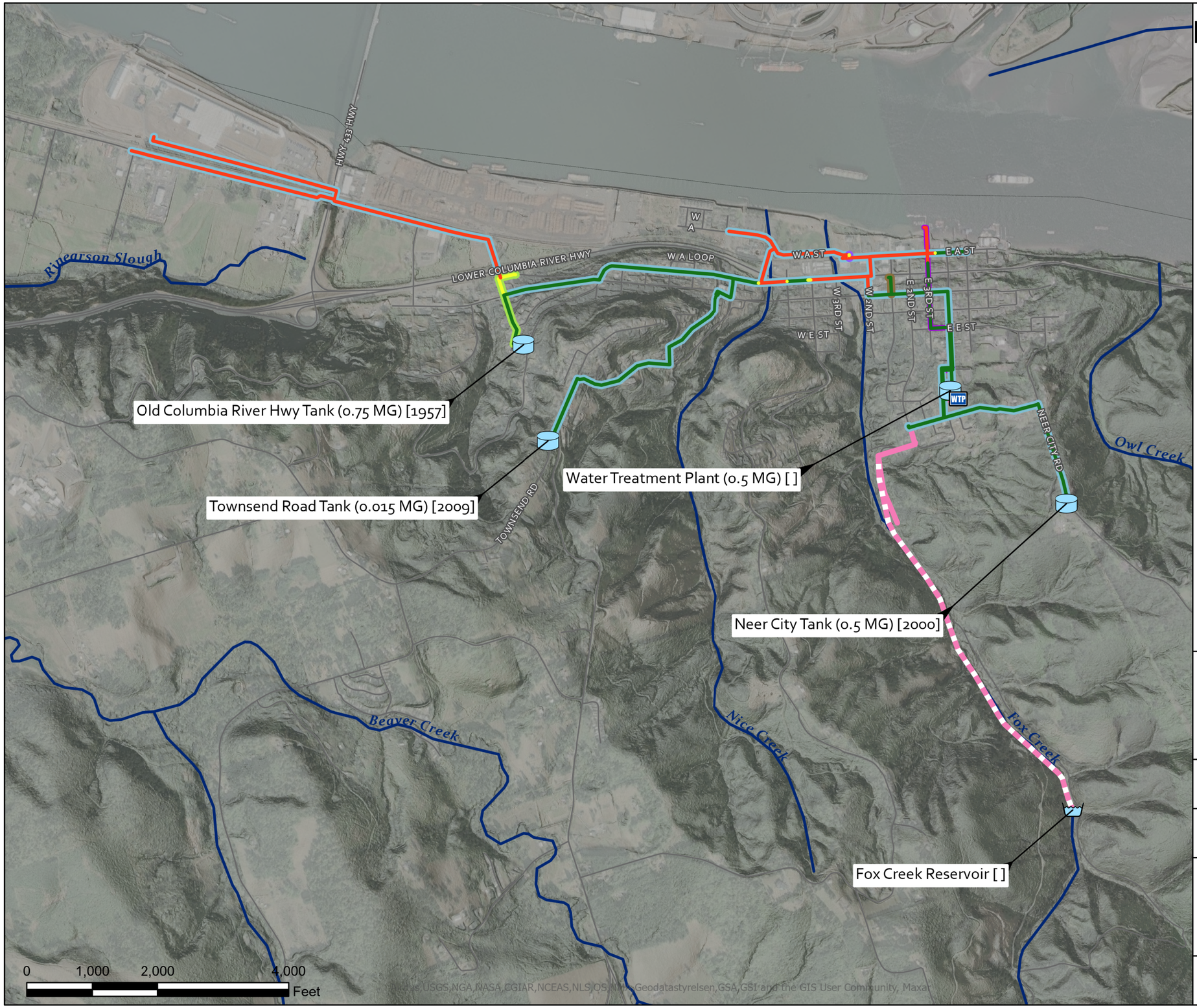
Table 5-3. Estimated Breaks Due to Shaking by Pipeline Material in Columbia Raw Water Pipelines

Material Description	Symbol	Maximum Diameter, in	Pipeline Length, ft	Estimated Breaks by PGV	Estimated Breaks by PGD
Cast iron	CI	12	4,373	0.0	3.4
		Total	**4,373**	**0.0**	**3.4**

Note: Estimated breaks are back calculated from RR per pipeline segment length.

The results of the analyses indicate that pipe damage due to ground shaking is low. However, pipelines located in areas of high liquefaction and large lateral spreading displacement (or permanent ground deformation) will suffer more damage. This includes areas near the Columbia River and in some localized drainage areas. A map of the City Pipeline Fragility with repair rates across the backbone system is shown on Figure 5-2.

The findings of the City pipeline fragility evaluation are summarized in the geotechnical seismic hazards evaluation report in Appendix A.



Legend

Rainier Water System

- Fox Creek Reservoir
- Existing Tank
- Water Treatment Plant
- Columbia River Pipeline
- Fox Creek Pipeline
- Fox Creek, Unknown Alignment

Material

- Cast Iron (<=12")
- Ductile Iron (<=12")
- Galvanized (<=12")
- Unknown (<=12")

Fragility Backbone Repair Rate

RR PGD ALA Repair Rate

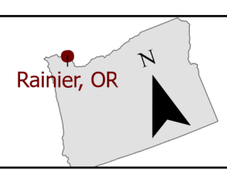
- < 1 (breaks/1000 feet)
- 1 - 4 (breaks/1000 feet)
- 4 - 6 (breaks/1000 feet)

NOTES:
SCALE: 1:18,000

DATA SHOWN ARE BASED ON DATA FROM DOGAMI Open File Report O-20-01 and ODGC7 AND HAVE NOT BEEN VERIFIED.

RAINIER WATER SYSTEM
SEISMIC RESILIENCE EVALUATION

PIPELINE FRAGILITY MAP



McMILLEN JACOBS ASSOCIATES

APR 2023

FIGURE 5-2

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

- Pipe replacement: Replace existing AC and CI pipes with more seismic resilient pipeline systems (lower break rates) such as welded steel pipe, DI pipe with restrained joints, Earthquake Resistant Ductile Iron Pipe (ERDIP), or high-density polyethylene (HDPE) pipe (AWWA-C906) or Molecularly Oriented PVC pipe (AWWA-C909).
- Assess local conditions where pipes cross minor drainages.
- Develop Seismic Pipeline Standards for design of future pipelines.

Based on the findings of the seismic hazard evaluation, a list of upgrades or improvements has been developed for the backbone pipelines based on their criticality. Table 5-4 shows the most critical projects and Table 5-5 shows the least critical projects.

Table 5-4. Most Critical Recommended Projects

RED Repair Rate Most Critical	Project Description	Cost Estimate, dollars
Existing raw water intake pipe[(a)]	Replace approximately 300 feet of existing 10-inch CI pipe from E B St to the Columbia River intake with restrained 10-inch DI pipe	200,000
E/W A St – Existing 10-inch and 12-inch pipe[(a)]	Replace approximately 800 feet of existing 10-inch CI pipe and approximately 2,000 feet of existing 12-inch CI pipe with restrained 12-inch DI pipe	350,000
W B St, W 2nd Street and Veterans Way – Existing 12-inch pipe[(a)]	Replace approximately 2,500 feet of existing 12-inch CI pipe with restrained 12-inch DI pipe	300,000
Mill St– Existing 10-inch pipe[(a)]	Replace approximately 350 feet of existing 10-inch galvanized pipe with restrained 12-inch DI pipe	55,000

(a) Indicates the project is included in the Capital Improvement Projects list in Chapter 6.

Table 5-5. Least Critical Recommended Projects

GREEN Repair Rate Least Critical	Project Description	Cost Estimate, dollars
Raw water line from E A Street to the Water Treatment Plant – Existing 10-inch pipe	Replace approximately 2,350 feet of existing 10-inch CI and DI pipe with restrained 10-inch DI pipe	260,000
E 1st between W B St and W C Street – Existing 12-inch pipe	Replace approximately 260 feet of existing 12-inch pipe (material unknown) with restrained 12-inch DI pipe	40,000
E C St from W 2nd Street to E 4th Street then along E 4th Street to the water treatment plant – Existing 12-inch pipe	Replace approximately 2,600 feet of existing 12-inch CI pipe with restrained 12-inch DI pipe	370,000
Pipeline from the water treatment plant to Neer City Reservoir – Existing 12-inch pipe	Replace approximately 3,400 feet of existing 12-inch CI pipe with restrained 12-inch DI pipe	485,000
E Norwood St from Rainier Blvd to E 2nd Street – Existing 12-inch pipe	Replace approximately 550 feet of existing 12-inch CI pipe with restrained 12-inch DI pipe	80,000
Existing 12-inch galvanized pipe to the Old Columbia River Highway Tank	Replace approximately 800 feet of existing 12-inch galvanized with restrained 12-inch DI pipe	115,000
Washington Way and W B Street – Existing 12-inch pipe	Replace approximately 4,000 feet of existing 12-inch CI pipe with restrained 12-inch DI pipe	570,000
Existing 6-inch CI pipe along Fern Hill Road from W B Street to the Townsend Road Tank	Replace approximately 4,500 feet of existing 6-inch CI pipe with restrained 6-inch DI pipe	405,000

CHAPTER 6
Water System Capital Improvement Program

This chapter presents the recommended CIP for the City's existing and future water system based on the evaluations described in Chapter 4 and Chapter 5 of this WMP Update. The chapter provides a summary of the recommended capital improvement projects, along with estimates for probable construction costs. Probable construction cost estimates are developed individually for each proposed improvement project.

It should be noted that the recommended CIP only identifies improvements at a Master Plan level and does not necessarily include all required on-site infrastructure or provide design of improvements. Subsequent detailed design is required to determine the exact sizes and locations of these recommended improvements.

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

- Cost Estimating Assumptions
- Recommended Capital Improvement Program

6.1 COST ESTIMATING ASSUMPTIONS

Construction costs are presented in September 2021 dollars based on an Engineering News Record (ENR) Construction Cost Index (CCI) of 12,464 (20-Cities Average). Construction costs were developed based on a combination of RSMeans base construction costs for typical water utility construction activities, and construction costs previously estimated by West Yost for similar facilities in Oregon with similar CCI. An estimating contingency of 30 percent of the base construction costs is used. Markups for engineering, legal, and administrative services (ELA) during design and construction are 25 percent of the base construction costs plus the final contingency, as listed below.

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

The total CIP cost mark-ups equal to 62.5 percent of the estimated base construction costs[1]. For this WMP Update, it is assumed that recommended distribution system facilities will be developed in public rights-of-way or on public property; therefore, land acquisition costs have not been included. The construction cost estimates do not include costs for annual O&M. A summary of the construction cost assumptions for pipeline and storage improvements are included below.

6.1.1 Pipelines

Table 6-1 presents the unit construction costs for water pipelines 6-inches through 12-inches in diameter. These unit costs are for pipeline construction in developed areas and are representative of pipeline construction conducted under common or normal conditions. These unit costs could be significantly higher under special or difficult conditions. The unit construction costs presented below generally include pipeline materials, trenching, placing, and jointing pipe, valves, fittings, hydrants, service connections, placing imported pipe bedding, native backfill material, and asphalt pavement replacement, if required.

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

Table 6-1. Unit Construction Costs for Pipelines[a]	
Pipeline Diameter, inches	Unit Construction Cost, $/linear foot
6	80
8	90
10	100
12	110
(a) Based on September 2021 ENR CCI of 12,464 (20-Cities Average).	

6.1.2 Storage Tanks

Table 6-2 presents the estimated construction cost of two storage tanks sized to match recommendations in *Chapter 4 Water System Analysis.* Storage tank costs were extrapolated from the RSMeans base construction costs for steel potable water storage tanks. This method accounts for fixed costs, such as site preparation and foundation, that contribute to a larger portion of the total construction costs for smaller tanks.

Table 6-2.Construction Costs for New Storage Tanks[a]	
Storage Capacity, gallons	Construction Cost, dollars
80,000	301,000
130,000	351,000
(a) Based on September 2021 ENR CCI of 12,464 (20-Cities Average).	

6.2 RECOMMENDED CAPITAL IMPROVEMENT PROGRAM

This section presents a summary of the capital improvement program recommended to address identified deficiencies. Recommended capital improvement projects were identified as capacity improvements, supplemental studies, and seismic improvements. Capacity improvements include pumping, storage, and pipeline projects. Pipeline projects are further categorized as Small Diameter Mains Improvements and Redundancy and Reliability Improvements. The location of proposed capacity improvements are shown on Figure 4-4 (Chapter 4). A follow-up study has also been included in the recommended CIP to perform a seismic, geotechnical, structural, and mechanical assessment of the raw water supply facilities, reservoirs, and WTP, as well as evaluate any improvements that would be required to utilize Fox Creek as a water supply. Seismic improvements identified in *Chapter 5 Seismic Risk and Mitigation Plan* are also included in the capital improvement program. Some projects are deemed higher priority improvements and are identified as 5-year capital improvements.

Table 6-3 presents the recommended 20-year CIP, which has a total estimated capital cost of $54,713,000. A 5-year CIP is also presented on Table 6-3, with an estimated capital cost of $1,544,000. Pipeline replacements under the Small Diameter Mains Improvements should also be prioritized annually, at a projected cost of approximately $137,850 per year. It is recommended that the City accounts for future inflation by increasing the costs by 3 percent per year from 2021 dollars during preparation of the annual budget.

6.2.1 Long-Term Water Supply Improvements

Previously discussed in Chapter 5, the City's water supply infrastructure at the Columbia River and Fox Creek are both susceptible to various types of seismic hazards. Two alternatives were developed to improve the quality and redundancy of the City's existing water supplies, shown as WS-1 and WS-2 in Table 6-3. The total capital improvements cost of approximately $13.5 million assumes that both water supply alternatives are constructed, however, constructing either alternative would create a seismically resilient water supply source. Rehabilitation of the Fox Creek water supply is estimated at $6 million and provides the City with a backup water supply if one is damaged in an earthquake. Alternatively, reconstructing the Columbia River intake facility at a different location is estimated at $7.5 million and decreases the likelihood of failure of the Columbia River supply in an earthquake; however, the City would have no water supply if failure does occur or if the Columbia river is unsafe as a water supply after an earthquake. It is recommended that the City implement either WS-1 or WS-2, at a minimum, to address the seismic hazards identified in Chapter 5.

Table 6-3. Recommended 20-Year CIP[a]

Improvement Type	Priority	Project ID	Improvement Description	Estimated Construction Cost[b]	Total Capital Cost (includes mark-ups)[b,c]
Referenced Maintenance					
Drainage Improvements at Old Columbia Hwy Reservoir	Annual	M-1	New drainage improvements should be implemented to reduce the risk of slope instability above the reservoir; if not mitigated, this issue may have the potential to trigger a large-scale landslide and impact the reservoir.	-	-
Columbia River Raw Water Intake and Pump Station	Annual	M-2	Address structural deficiencies listed in Chapter 5, Section 5.3.1.2.	-	-
Neer City Reservoir	Annual	M-3	Bolt tank to the concrete foundation; Clean and touch-up paint to prevent loss of steel thickness	-	-
Old Columbia Highway Reservoir	Annual	M-4	Bolt tank to the concrete foundation; Clean and touch-up paint to prevent loss of steel thickness	-	-
Water Treatment Plant - Building	Annual	M-5	Address structural deficiencies listed in Chapter 5, Section 5.3.2.2.	-	-
Water Treatment Plant - Clearwell	Annual	M-6	Bolt tank to the concrete foundation; Clean and touch-up paint to prevent loss of steel thickness	-	-
General Non-Structural Maintenance	Annual	M-7	Review and address the best-practices described in Chapter 5, Section 5.4. This is assumed to be an ongoing program over 20 years.	-	-
			Maintenance Subtotal:	**$50,000**	$81,000
Capacity Improvements					
Pipelines - Small Diameter Mains Improvements					
6-inch Diameter	20-year	--	Replace 10,200' of existing pipeline with 6-inch pipeline	$818,000	$1,329,000
8-inch Diameter	20-year	--	Replace 9,500' of existing pipeline with 8-inch pipeline	$879,000	$1,428,000
			Small Diameter Main Improvements Subtotal:	**$1,697,000**	**$2,757,000**
Pipelines - Redundancy and Reliability Improvements					
10-inch Diameter	5-year	D-1	Replace 1,100' of existing 4-inch pipeline with 10-inch along B St from W Second St to W Fourth St; Construct 400' of new 10-inch pipeline along A St between W Fourth St and W Fifth St	$150,000	$244,000
8-inch Diameter	20-year	D-2	Replace 1,200' of 4-inch pipeline with 8-inch along East B St from W First St to E Fifth St; Tie into crossing 6-inch pipeline at the intersection of E Third St and East B St	$103,000	$167,000
10-inch Diameter and PRV	20-year	D-3	Replace 1500' of 4-inch and 8-inch pipeline with 10-inch pipeline Construct 1,100' of new 10-inch pipeline with PRV along Neer City Road from existing tee blind flange to East F St	$358,000	$582,000
12-inch Diameter	20-year	D-4	Replace 1,400' of 4-inch pipeline with 12-inch along Washington Way from Mill St to Rock Crest St; Construct 530' of new 12-inch pipeline along Rock Crest St, crossing Hwy 30 to existing 12-inch	$212,000	$345,000
8-inch Diameter and PRV	5-year	D-5	Replace 1,700' of 4-inch and 6-inch pipeline with 8-inch along W Fourth St from West B St to West F St; Construct 600' of new 8-inch pipeline with PRV along Debast Rd from West F St to existing dead end on Debast Rd	$310,000	$504,000
8-inch Diameter	5-year	D-6	Replace 171' of 4-inch pipeline with 8-inch along W Third St; Construct 350' of new 8-inch pipeline along W Third St from West C St to West E St; Tie into existing pipelines on West C St, West E St, and West E St	$47,000	$76,000
8-inch Diameter	20-year	D-7	Replace 6,700' of 6-inch pipeline with 8-inch along Maple Dr, Fern Hill Rd, Riverview Dr, and Townsend Rd	$601,000	$977,000
8-inch Diameter	20-year	D-8	Replace 1,200' of 2-inch pipeline with 8-inch along E E St From E Fourth to E Third, and south along E Third St; Construct 100' of new 8-inch pipeline from dead-end on E Third St to the dead-end on E Second St	$116,000	$189,000
12-inch Diameter	20-year	D-10	Construct 1,500' of 12-inch pipeline along Washington Way between Mill St and W B St	$163,000	$265,000
			Redundancy and Reliability Improvements Subtotal:	**$2,060,000**	**$3,349,000**
Storage Improvements					
New Storage Tank	5-year	R-1	Demolish existing redwood tank and replace with new a new 80,000 gallon tank	$301,000	$489,000
New Storage Tank	20-year	R-2	Construct a new 130,000 gallon tank parallel to the Neer City Reservoir	$351,000	$570,000
			Storage Improvements Subtotal:	**$652,000**	**$1,059,000**
Pumping Improvements					
New Fire Pump	20-year	P-1	Replace existing Old Highway fire pump with pump of greater than or equal to 1,000 gpm capacity for fire flow	$80,000	$130,000
New Hydropneumatic Tank	5-year	P-2	Replace the existing problematic hydropneumatic tank at the Old Highway Pump Station.	$65,000	$106,000
			Pumping Improvements Subtotal:	**$145,000**	**$236,000**
			Capacity Improvements Subtotal:	**$4,554,000**	**$7,401,000**

Table 6-3. Recommended 20-Year CIP[a]

Improvement Type	Priority	Project ID	Improvement Description	Estimated Construction Cost[b]	Total Capital Cost (includes mark-ups)[b,c]
Supplemental Studies					
Seismic and Reliability Follow-up Study	5-year	S-1	Site-specific geotechnical and mechanical investigations as part of a seismic evaluation of the reservoirs and other structures in the system.	--	$125,000
Long Term Resilient Water Supply Detailed Evaluation	5-year	S-2	Perform geotechnical and stability analyses, hydraulic and dam break analyses for the Fox Creek dam, and evaluation of the improvements needed to treat Fox Creek raw water. Perform evaluation of a new site for relocation of the Columbia River intake pump	--	$150,000
			Supplemental Studies Subtotal:	**--**	**$275,000**
Seismic Improvements[e]					
12-inch and 10-inch Restrained Ductile Iron	20-year	E-1	Replace 2,000' of 12-inch cast iron pipeline with 12-inch ductile iron pipeline along West A St; Replace 800' of 10-inch cast iron pipeline with 10-inch ductile iron pipeline along West A St	$350,000	$569,000
12-inch Restrained Ductile Iron	20-year	E-2	Replace 2,500' of 12-inch cast iron pipeline with 12-inch ductile iron pipeline along Veterans Way, West B St, and W Second St	$300,000	$488,000
10-inch Restrained Ductile Iron	20-year	E-3	Replace approximately 300 feet of existing 10-inch cast iron pipe from E B St to the Columbia River intake with restrained 10-inch ductile iron pipe.	$200,000	$325,000
12-inch Restrained Ductile Iron	20-year	E-4	Replace approximately 350 feet of existing 10-inch galvanized steel pipe along Mill St with restrained 12-inch ductile iron pipe.	$55,000	$89,000
			Seismic Improvements Subtotal:	**$905,000**	**$1,471,000**
Water Supply Reliability					
Columbia River Raw Water Intake and Pump Station	20-year	WS-1	Rebuild the raw water intake facility on a site not susceptible to liquefaction and lateral spreading hazards.	--	$7,500,000
Fox Creek Dam Rehabilitation	20-year	WS-2	Rehabilitate the existing Fox Creek Dam to create a reliable back-up water supply to the Columbia River.	--	$6,000,000
			Water Supply Reliability Subtotal:	**$905,000**	**$14,971,000**
			5-year Capital Improvement Program Subtotal:	**$873,000**	**$1,694,000**
			20-year Capital Improvement Program Subtotal:	**$4,586,000**	**$20,953,000**
			Capital Improvement Program Total:	**$5,509,000**	**$22,728,000**

(a) Costs are based on September 2021 ENR CCI of 12,464 (20-Cities Average).
(b) Totals rounded to nearest $100.
(c) Costs include mark-ups equal to 63 percent ([30 percent x 25 percent]; Contingency = 30 percent of construction costs; Engineering, Legal, Administration (ELA) = 25 percent of construction costs times the Contingency).
(d) Pressure reducing valves are assumed a construction cost of $75,000.
(e) Seismic improvements are high-priority improvements identified in *Chapter 5 Seismic Risk and Mitigation Plan* for a 20-year planning period. See Chapter 5 for additional recommended seismic improvements.

Appendix A

Technical Memorandums By Delve Underground (formerly known as McMillen Jacobs Associates)

A1: Seismic Hazards Evaluation – October 21, 2022

A2: Site-Specific Seismic Hazards Evaluation (Columbia River Intake Facility and Water Treatment Plant) – April 27, 2023

Technical Memorandum

To:	Sandrine Ganry, PE, West Yost Associates, Inc.	Project:	City of Rainier Water Master Plan Update
From:	Wolfe Lang, PE, GE, & Devin Roth, EIT	cc:	
Date:	October 21, 2022	Job No.:	6219.0
Subject:	Seismic Hazards Evaluation		

1.0 Introduction

The City of Rainier, Oregon (City) is updating their water system master plans. The City has contracted West Yost Associates, Inc. (West Yost) to provide professional services for the master plan updates and resilience study. As part of the study, a seismic resiliency evaluation of the water system is required in conjunction with the Oregon Resilience Plan (ORP). West Yost has retained McMillen Jacobs Associates (McMillen Jacobs) to conduct seismic hazard evaluation and backbone pipe vulnerability assessment.

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

1. Review of DOGAMI seismic hazard maps for a magnitude 9.0 CSZ event in the City's service area;
2. Review of available geological information;
3. Review of available well log information to verify DOGAMI seismic hazard maps;
4. Site reconnaissance to address key geological and geotechnical assumptions and to examine areas that are potentially prone to failures from lateral spreading and seismic landslide hazards;
5. Develop estimates of strong ground shaking, liquefaction-induced settlement, lateral spreading displacement, seismic landslide slope instability, and develop maps illustrating these hazards in relation to the City's backbone system;
6. Conduct pipe fragility analysis for the backbone pipe system under strong ground shaking and seismic ground deformation, and;
7. Develop this memorandum summarizing the results of our evaluations and including seismic hazards and fragility maps.

McMillen Jacobs completed these tasks at the identified critical water facilities and backbone pipes as shown on Figures 1 to 6. In the following sections, we present the results of the data review, seismic hazards evaluation, pipe fragility assessment, and a summary of geotechnical hazards along the backbone system.

2.0 Background Information Review and Site Reconnaissance

McMillen Jacobs performed background information review and site reconnaissance in the vicinity of Rainier, Oregon. These tasks included review of mapped seismic hazards, geologic and seismic hazards literature, historic well logs, various construction drawings and reports for the Rainier Water System and in adjacent areas.

2.1 Geologic Setting

The City of Rainier, Oregon lies along the southwest bank of the Columbia River and at the eastern foot of the northern Coast Range. The Coast Range is composed predominantly of early Eocene to middle Miocene (about 50 to 15 million years before the present) marine sedimentary rocks, with some intruded basaltic lava and flow breccia that are the remnants of ancient volcanic islands. The Coast Range sediments and the intrusive volcanic rocks were warped upward above sea level at the leading edge of the continent when compressed between the colliding continental and oceanic crustal plates. Portions of the rising eastern flank of the Coast Range were covered by younger (middle Miocene, about 17 to 15 million years before present) lava flows of the Columbia River Basalt Group (CRBG), which were then uplifted with the rest of the Coast Range.

Geologic units that underlie the Rainier, Oregon area can be divided into three informal units based upon their source and genesis: volcanic rocks, terrestrial sedimentary rocks, and alluvial sediments. Each of these three units is briefly described below from oldest to youngest.

Volcanic Rock Unit
The Volcanic Rock Unit is the oldest unit and is partially exposed at the ground surface west of Rainier along Highway 30 and forms the bluff facing the Lewis and Clark Bridge. The unit consists largely of basalt lava flows, flow breccia, and other fragmented volcanic ejecta. Individual lava flows are commonly interbedded with layers of sandstone and pebble-cobble conglomerate composed of eroded volcanic materials and shallow marine sedimentary rocks. Surface soils consist of gravelly silt colluvium and grade with depth to weathered rock.

Terrestrial Sedimentary Rock Unit
The Terrestrial Sedimentary Rock Unit is also composed largely of volcanic rock, but contains fewer lava flows and more clastic volcanic material and sedimentary deposits (also derived from volcanic deposits). This unit underlies most of the Rainier town center and residential areas from Highway 30 south. Below an elevation of about 500 feet, this unit is dominated by stratified sandstone, siltstone, shale, and conglomerate, with occasional interbedded lava flows, tuffs, and tuff breccia. Surface soils are generally greater than feet thick and consist of silty colluvium and residuum derived from the underlying bedrock with the addition of a thin cover of wind-blown silt over the peaks and ridge tops. The Old Columbia River Highway Tank, Townsend Road Tank, and the Water Treatment Plant and tank are all located over this rock unit. Above elevations of about 500 feet this unit contains a mix of lava flow rock sedimentary rock. The Neer City Tank sits at approximately elevation 550 feet. The Fox Creek Reservoir is near an elevation of 440 feet.

Alluvial Sediments Unit

Alluvial sediments lie in the main Columbia River channel and underlie the historical floodplain, which generally is the area between Highway 30 and the shoreline. In general, the sediment in the main channel of the river and deeper alluvial layers beneath the flood plains on both sides of the river consists of dense to very dense sandy gravel with cobbles and boulders. Composition of the flood gravel is dominantly fresh, hard basalt. In the Columbia River channel, these gravel deposits are present beneath a 3- to 6-foot-thick layer of loose sand.

Beneath the floodplain on the Rainier side of the river, stratified loose to medium dense clay, silt, sand, and silty sand overlie the gravel deposits. The fine-grained floodplain deposits vary from zero to about 90 feet thick, being thickest near the river and thinning to zero near Highway 30. Dredged sand fill has raised and extended the broadened floodplain to support industrial and marine shipping facilities.

2.2 Subsurface Data

McMillen Jacobs reviewed published geologic maps and available subsurface condition information in the vicinity of the water system. There was little geotechnical boring and site-specific geotechnical exploration data available for the water system service area, so we used well logs from the Oregon Water Resource Department's (ORWD) database. Where possible, we used this data to confirm or revise the extents of mapped geologic and hazard layers from the DOGAMI O-20-1 and to conduct soil liquefaction potential evaluation.

Our research found at least 60 documented wells in the Alluvial Sediments Unit. Of these, 12 logs were sufficient detailed to provide an estimated range of depth non-liquefiable material, typically described as gravel. documents containing useful geotechnical information. We show a summary of these documents in Table 1.

Table 1: ORWD Well Logs

	ORWD Well Log	Geologic Unit	Boring Depth (Feet bgs)	Depth to Gravel (Ft bgs)	GWT (Feet bgs)
1	COLU_1914	Alluvial Sediments	110	41	12
2	COLU_3828	Alluvial Sediments	60	30	8
5	COLU_263	Alluvial Sediments	36	26	11
4	COLU_51170	Alluvial Sediments	118	55	27
6	COLU_54001	Alluvial Sediments	77	30	-
7	COLU_57	Alluvial Sediments	60	25	25
8	COLU_52557	Alluvial Sediments	76	20	51
9	COLU_1909	Alluvial Sediments	145	40	12
10	COLU_55080	Alluvial Sediments	100	90	-
11	COLU_54659	Alluvial Sediments	60	45	-
12	COLU_1911	Alluvial Sediments	375	45	-

2.3 Site Reconnaissance

On February 4th, 2021 Devin Roth and Wolfe Lang of McMillen Jacobs met with Sandrine Ganry of West Yost and performed a geotechnical reconnaissance for the following water facilities of the City:

- Columbia River Intake;
- Rainier Water Treatment Plant;
- Neer City Tank (0.5 MG);
- Old Columbia River Hwy Tank (0.75 MG);
- Townsend Road Tank (0.015 MG).

These facilities are shown in Figures 1 through 6 and are part of the critical facilities in the Seismic backbone system. During the reconnaissance, we noted site conditions, surface or exposed soil and rock conditions, site topography, proximity to bodies of water and steep slopes. Selected photographs from the site visits are provided in Appendix A. A summary of suggestions based on the site visit and subsequent data review is presented in Section 5.

3.0 Seismic Hazard Evaluation and Mapping

Recent earthquakes in Japan, New Zealand, Chile and elsewhere, and an increased understanding of the Cascadia Subduction Zone (CSZ), have increased the recognition of the earthquake hazard in Oregon. In 2011, Oregon legislature passed a resolution directing the Oregon Seismic Safety Policy Advisory Commission (OSSPAC) to prepare the Oregon Resilience Plan (ORP). The purpose of the ORP is to set policy direction for protecting lives and maintaining economic and commercial activity following a magnitude 9.0 CSZ earthquake (Oregon Resilience Plan 2013). In 2020 DOGAMI published Open File Report O-20-1, which includes updated hazard maps for Columbia County, Oregon (DOGAMI 2020).

Recent studies indicate that there have been numerous large-magnitude earthquakes generated from the CSZ (Goldfinger et al., 2012). These records extend as far back as approximately 10,000 years and indicate an average recurrence of about 500 to 530 years for great earthquakes (larger than M 8.0) that rupture along the entire length of the CSZ (from Cape Mendocino, California to Northern Vancouver Island, British Columbia). The previous CSZ earthquake occurred in January 1700 (Satake, et al., 1996) and is estimated to have been a M 9.0 event. Studies by the USGS (Goldfinger et al., 2012; OSSPAC, 2013) indicate that there is a 7 to 12 percent probability of a great CSZ event impacting the entire Pacific Northwest region. A more recent study by University of Oregon (Goldfinger et al., 2016) estimates that a CSZ earthquake with a magnitude greater than 8.5 has a probability of occurrence on the order of 16 to 22 percent over the next 50 years.

Earthquake hazards within the Project area include earthquakes generated from several sources:

- Crustal faults;
- Within the deep subducted portion of the Juan de Fuca Plate, along the CSZ (referred to as intraslab sources); and
- Along the locked zone of the Cascadia Subduction Zone fault interface capable of producing great, megathrust events.

Large subduction zone earthquakes are characterized by a long duration of significant ground shaking. The 2010 subduction zone earthquake in Maule, Chile resulted in approximately 100 seconds of significant ground shaking. The 2011 subduction zone earthquake in Tohoku, Japan resulted in between about 2 to 3 minutes of significant ground shaking. For comparison, crustal earthquakes with similar seismogenic characteristics exhibited significant ground shaking for about 10 to 20 seconds (specifically the 2010-2011 Christ Church, New Zealand sequences, 1999 Chi-Chi, Taiwan, and 1983 Coalinga, California earthquakes).

Due to the long duration of ground shaking, a CSZ earthquake is expected to cause higher levels of permanent ground deformation (PGD) than crustal and intraslab sources. Permanent ground deformations pose critical seismic risks to the City's water system seismic backbone.

In some instances, for structures and facilities (pump stations, treatment plants, reservoir, dam), the seismic design criteria are governed by other code-based procedures (ASCE 7-16, OSSC 2014, Oregon Dam Safety). Typically, these code-based procedures require use of the Maximum Considered Earthquake (MCE) ground motion. The MCE ground motion is derived from USGS Seismic Hazard Maps and represents the most server earthquake effects. The MCE is a probabilistically derived composite event that is aggregated from all potential earthquake sources that could impact a site and having a return interval of 2,475 years. For comparison, the ORP M9.0 scenario is roughly analogous to a 500-year return interval event.

As part of the ORP, The OSSPAC created a Cascadia Earthquake Scenario workgroup which was charged with developing a description of the likely ground motions (velocity and accelerations) and permanent ground deformations (PGD) to be expected from a M9.0 CSZ event. The workgroup, along with the Oregon Department of Geology and Mineral Industries (DOGAMI), developed M9.0 earthquake scenario maps, which are included in the DOGAMI Open-File Report O-13-06. In 2020, DOGAMI released updated hazard maps of Columbia County, Oregon, (DOGAMI 2020) Of primary interest in these earthquake scenario maps are the following seismic hazards, which formed the baseline levels in our evaluations of seismic hazards:

- Peak Ground Velocity (DOGAMI O-13-06 Plate 2)
- Liquefaction Probability (DOGAMI O-13-06 Plate 3)
- Lateral Spreading PGD (DOGAMI O-13-06 Plate 4)
- Seismic Landslide PGD (DOGAMI O-13-06 Plate 6)

We used the M9.0 CSZ event as the basis of our analyses to be consistent with the ORP and the City's approach. Seismic hazards that pose the most risk to the City's water system include strong ground shaking (peak ground velocity and accelerations), liquefaction, lateral spreading, and seismic-induced landslides. In our hazard evaluations, we overlaid the City's seismic backbone onto the DOGAMI maps as a starting point. As discussed subsequently, we refined and updated the DOGAMI maps in areas that we found appropriate based on field observations, subsurface data and geotechnical engineering analysis.

3.1 Geologic Map

The Geologic Map (Figure 1) describes the mapped surficial geologic units in the vicinity of the Rainier Water System backbone and is based on the DOGAMI Oregon Geologic Data Compilation Release 7 (OGDC-7). This mapped units correspond to the generalized descriptions discussed in Section 2.1. As shown on Figure 1, much of the backbone infrastructure at low elevations is located within recent alluvial sediments. At higher elevations shallow basalt and terrestrial sediments are prevalent. Geologic conditions in smaller drainages such as Fox Creek are not mapped, however, they will be shown to behave more like the alluvial deposits under seismic loads in the subsequent figures.

3.2 Peak Ground Velocity

The intensity of ground shaking at a site is known as peak ground velocity (PGV). The magnitude of PGV is dependent on the magnitude and distance from the seismic source, and the ground material through which seismic waves pass. The rapid and extreme shaking during an earthquake can cause transient stress and strain in pipelines that can be damaging if the pipe material and joints are not strong enough to withstand the transient ground deformations. Damage from ground shaking occurs even when there is no permanent ground deformation.

The Peak Ground Velocity Map (Figure 2) shows estimated PGV ranges in the Rainier area for a CSZ 9.0 event. The map generally corresponds to areas with the mapped geology of the area. Consolidated soils and rock such as volcanic or sedimented deposits will experience velocities between 7 and 11 inches per second. Regions where the soils are less consolidated or disturbed, such as recent alluvial deposit (from the Columbia River or smaller drainages), landslide deposits, or fill will experience higher velocities, up to 21 inches per second. These extents approximately correspond to the soil units show in Figure 1, with higher velocities also shown in small drainage channels and landslide deposits.

3.3 Liquefaction Settlement

Liquefaction is a phenomenon in which ground shaking from an earthquake transforms soil from a solid state to a viscous fluid state. Soils that are susceptible to liquefaction are generally sands and non-plastic to low-plastic silts that are saturated (below groundwater level). Silts and silty soils with a plasticity index less than 7 are generally considered to be susceptible to liquefaction. The results of soil liquefaction include loss of shear strength, loss of soil materials through sand boils or flow, flotation of buried chambers/pipes, and post-liquefaction reconsolidation (settlement).

To assess the liquefaction potential in the area, we first reviewed the published DOGAMI Liquefaction Probability map which indicates high liquefaction potential in low-lying areas near the Columbia River, and other smaller drainages, such as Fox Creek, where moderate liquefaction potential is present. These assessments are based on the assumed liquefaction susceptibility for the mapped soil types, assumed groundwater depths, and ground shaking magnitude. Where shallow rock exists, there is no settlement hazard. In other areas, historic well logs (Table 1) show a potentially liquefiable layer from 15 to 45 feet thick in the where a liquefaction hazard exists, vertical settlement of 6 to 12 inches could occur. This value is based on a volumetric settlement of 2 – 4%. This is a conservative estimate and could be revised with site specific geotechnical investigation and soil lab analysis.

3.4 Lateral Spreading

Liquefaction can result in progressive ground deformation known as lateral spreading. Lateral spreading generally occurs along river/creek banks and within sloping ground areas. The lateral movement and loss of support of the liquefied soil breaks the overlying non-liquefied soil "crust" into blocks that progressively move downslope or toward a free face in response to the earthquake generated ground accelerations. Each cycle of loading from the earthquake incrementally pushes these blocks downslope. The potential and magnitude of lateral spreading depends on the liquefaction potential of the soil, the magnitude and duration of earthquake ground accelerations, the site topography, and the post-liquefaction strength of the soil. Lateral spreading can result in both vertical and horizontal components of PGD, but for discussion purposes and this screening-level of analysis, the reported estimates of PGD can be considered horizontal. The Liquefaction Lateral Spreading Map (Figure 4) shows the location of the water system backbone compared to seismic liquefaction lateral spread hazards.

Within the study area, we used historic well logs to confirm that the DOGAMI liquefaction lateral spreading estimates are reasonable. The areas with high lateral spreading hazard general follow the Columbia River and smaller drainage channels. Lateral spreading PGD on the order of up to 15 feet are possible at this location. A site-specific lateral spreading study should be performed to characterize the hazard and develop mitigation strategies, as discussed in more detail later in this memo. Other areas founded on volcanic and sedimentary bedrock terrains are not at risk for lateral spreading.

3.5 Seismic Landslides

Earthquake induced landslides can occur on slopes due to the inertial force from an earthquake adding load to a slope. The ground movement due to landslides can be extremely large and damaging to pipelines and other structures. A map of the estimated levels of seismic landslide PGD is shown on Figure 5.

In general, the seismic landslide hazard along the backbone system is mapped as relatively low. However, along the Fox Creek raw water pipeline and near the Fox Creek Reservoir Dam, numerous landslides have been mapped by DOGAMI and are shown in Figure 5. The seismic landslide hazard is considered high in this area. Site-specific stability and resiliency evaluations will need to be conducted for this area to ensure seismic resilience.

4.0 Water System Backbone Fragility Evaluation and Mapping

McMillen Jacobs performed a seismic fragility evaluation to assess the vulnerability of the Rainier Water System backbone using the methods published in the report Seismic Fragility Formulations for Water Systems, American Lifelines Alliance, 2001 (ALA). The evaluation relies on the GIS data provided by the City and applies the seismic ground motions discussed above to produce a value called "repair rate" or RR in units of breaks per 1000-feet of pipe.

4.1 Seismic Fragility Methodology

The ALA method considers the primary seismic effects causing pipeline breakage to be ground shaking intensity (PGV) and permanent ground deformations (PGD). The capacity of a pipeline to resist these effects is determined by materials, size, joints and soil corrosivity. The effects from ground shaking are

mainly the transient ground movement by wave propagation and potential ground amplification, these effects are termed PGV, in units of inches per second. Seismic induced ground deformations can result from surface fault rupture, landslide, slope failure, soil liquefaction, lateral spreading, and differential settlement. For each section of pipe, the maximum of these is the PGD term, in units of inches. Typically, the transient load from ground shaking is generally low, and most earthquake damage to pipelines is due to PGD.

The ALA method provides two functions for estimating the likelihood of pipeline damage to a pipeline network in a seismic event, these methods are shown in Table 2. The fragility functions produce a result term called repair rate (RR) in units of breaks per 1000 feet of pipeline. These methods are empirical fragility functions developed by correlating the frequency of pipe damage, including leaks or breaks, which were severe enough to require repairs in past earthquakes and correlating with the ground shaking and PGD at the site of the repair. The K terms are a function of pipe properties, such as material and diameter, these can either increase or decrease the base pipe break rate.

Table 2: ALA Fragility Functions

Hazard	Vulnerability Functions	Lognormal Std Deviation, β	Comment
Wave Propagation	$RR = K_1 \times 0.00187 \times PGV$	1.15	Based on 81 data points of which largest percentage (38%) was for CI pipe.
Permanent Ground Deformation	$RR = K_2 \times 1.06 \times PGD^{0.319}$	0.74	Based on 42 data points of which the largest percentage (48%) was for AC pipe.

Notes: 1. Table derived from (American Lifelines Alliance 2001, Table 4-4)
2. RR = Repairs per 1,000 feet of main pipe.
3. K_1, K_2 = Material and size factors.
4. PGV = peak ground velocity, inches/second; PGD = Permanent ground deformation, inches.
5. PGD is the maximum value of landslide displacement or the resultant of lateral spreading and vertical settlement in inches.

4.2 Pipeline Fragility Evaluation

Applying the ALA fragility functions discussed above, McMillen Jacobs evaluated the fragility of the backbone pipelines and the Columbia River Raw Water Pipeline in the Rainier Water System. using GIS software to overlay mapped PGV and PGD hazards to the geolocated water system data provided by the City. The City did not provide a current alignment for the Fox Creek Alignment, so we did not perform a fragility analysis for this section. Table 3 summarizes pipeline breakage by driven by ground shaking driven (PGV) and ground deformation driven (PGD) for each pipe material in the system. The data show that the backbone system could expect to experience nearly 100 breaks or leaks requiring repairs.

The Rainier Water System backbone is composed of several different materials. Most of the backbone is cast iron (CI), with smaller sections consisting of ductile iron (DI), galvanized steel (GALV), steel (STL), and unknown (UNK) materials.

Table 3: Estimated breaks due to shaking by pipeline material in backbone

Material Description	Symbol	Max. Diameter (in)	Pipeline Length (ft)	Estimated Breaks by PGV	Estimated Breaks by PGD
Cast Iron	CI	12"	33172	1.0	90.3
Ductile Iron	DI	12"	49	0.0	0.1
Steel Small	GALV	12"	1504	0.0	2.1
Steel Large	STL	16"	608	0.0	2.5
Unknown	UNK	12"	291	0.0	0.0
	Total		**35624**	**1.0**	**95.0**

Note: Estimated breaks are back calculated from RR per pipeline segment length.

The same methods were applied to the Columbia Intake Raw Water Pipeline. There are approximately 4400 feet of CI pipe, with diameter 12" or smaller in the raw water system. Based on the analysis, we expect 3.4 breaks in the raw water system (summarized in Table 4). These breaks are localized in the raw water system north of E B St.

Table 4: Estimated breaks due to shaking by pipeline material in Columbia raw water pipelines

Material Description	Symbol	Max. Diameter (in)	Pipeline Length (ft)	Estimated Breaks by PGV	Estimated Breaks by PGD
Cast Iron	CI	12"	4373	0.0	3.4
	Total		**4373**	**0.0**	**3.4**

Note: Estimated breaks are back calculated from RR per pipeline segment length.

The results summarized above in Tables 3 and 4 are displayed in the Pipeline Fragility Map (Figure 6). This map compares pipeline material and repair rates across the backbone of the system. Based on the map, the areas of highest repair rates, with RR between 4 and 6 breaks per thousand feet of pipe, are the raw water pipeline north of E B St, and the distribution backbone lines that run along E/W A St, W B St, W 2nd St, Mill St, and Dike Rd.

The values used for PGV and PGD used for the fragility calculation are from the revised seismic hazards maps discussed in the previous section. For pipeline properties, we used the information from the City's GIS system. Typically, ductile iron (DI) pipes have better seismic performance than cast iron (CI) and asbestos concrete (AC) pipes (American Lifelines Alliance, 2001). However, regular DI pipe without joint restraint is not considered to be an earthquake resilient pipe material. Whereas welded steel pipe, HDPE pipes and DI pipes with earthquake resilient joints are considered to be earthquake resilient materials.

Our analyses results show that pipe damage due to ground shaking will be low. However, pipes in areas of large PGD, mainly due to liquefaction generated lateral spreading near Columbia River or drainages, will suffer more. This is consistent with observations of pipe systems after previous earthquakes. Historically most damage occurs as a result of large PGD.

The pipeline repair rates identified by the assessment include both leaks and breaks. For planning purposes, we recommend assuming that 50 percent of the total repairs consist of leaks and the remaining 50 percent consist of breaks. To improve the seismic resiliency of the backbone pipelines, we recommend the following pipe improvement elements according to their considered priorities:

- Within the scheduled resiliency improvement period suggested by ORP (in the next 45 years) systematically replace the AC and CI pipes with more seismic resilient pipe systems. Experiences and lessons learned from previous earthquakes show that flexible pipe systems, such as welded steel pipe, DI pipe with restrained joints, and high-density polyethylene (HDPE) pipe, had a much lower break rates than the more rigid and segmental DI, and concrete pipes.
- Assess the soil liquefaction and lateral spreading hazards at Columbia River Intake and raw water pipeline and improve the seismic performance according to the seismic ground deformation hazard level.
- Assess local conditions where pipes cross minor drainages. Some of these hazards may be overstated if the actual depth of the pipeline is below the region of liquefiable soil.

5.0 Seismic Hazard Assessment and Recommendations for Critical Facilities

In addition to the seismic hazard study for the overall service area, we conducted site visits to the Rainier Water Treatment Plan, Columbia River Intake, Neer City Tank, Old Columbia River Highway Tank, Townsend Road Tank. We attempted to visit the Fox Creek Reservoir, but the unpaved road to the site was blocked by fallen trees.

- **Water Treatment Plant** – The Rainier Water Treatment Plant sits in the hills to the south of downtown. The major structures at the plant are a 0.5 MG tank, a stilling basin, and 2 buildings which house filtration units and other mechanicals. Although the site is on a slope, the geologic map and nearby well logs indicate shallow rock in the area. We consider the liquefaction, lateral spreading, and landslide seismic hazards to be low at this site. We recommend that a geotechnical investigation be conducted at the site to confirm the subsurface condition and low seismic ground deformation hazards. Additionally, the seismic performance of the structures and mechanical components will need to be evaluated to ensure seismic resiliency under strong seismic ground shaking.

- **Columbia River Intake** – This facility is located at the bank of the Columbia River, adjacent to the municipal boat dock. The pump house sits on the riverbank at an elevation of 25 feet above sea level and 10 to 15 feet above typical river levels. The intake itself is located approximately 150 feet from shore, adjacent to the municipal boat dock. There is no existing geotechnical information for the site. The geologic map and nearby well logs indicate the presence of up to 50 feet of potentially liquefiable fine-to-coarse grained materials 50 feet bgs in the area, overlaying a harder gravel layer. We consider the liquefaction, lateral spreading, and landslide seismic hazards to be very high at this site. We recommend that a geotechnical and seismic study be conducted at the site to investigate the subsurface condition and seismic ground deformation hazards, and to

develop mitigation options. Additionally, the intake pipe to the river will need to be evaluated and a mitigation strategy developed to ensure seismic resiliency.

- **Neer City Tank** – This facility is a 0.5 MG water tank, constructed in 2000. The Neer City Tank is located in the hills to the southeast of the City. There is no existing geotechnical information available for review. Although there are steep slopes nearby, the geologic map and nearby well logs indicate shallow rock in the area. We consider the seismic hazards in terms of liquefaction, lateral spreading, and landslide to be low at this site. We recommend that a geotechnical investigation be conducted at the site to confirm the subsurface condition and low seismic ground deformation hazards. Additionally, the seismic performance of the reservoir structure will need to be evaluated to ensure seismic resiliency under strong seismic ground shaking.

- **Old Columbia River Highway Tank** – This facility is a 0.75 MG water tank, constructed in 1957. This tank is located in the hills to the west of downtown. There is no existing geotechnical information available for review. There is significant seepage from the slope to the east above the tank. There was a French drain installed at the site, but it is no longer functional. We recommend that this seepage issue be mitigated to reduce the risk of slope stability issue above the reservoir which may not only impact the nearby roadway and access reservoir and may have the potential in long-term to trigger a large-scale landslide to impact the reservoir. Although there are steep slopes nearby, the geologic map and nearby well logs indicate shallow rock in the area. We consider the seismic hazards in terms of liquefaction, lateral spreading, and landslide to be low at this site. We recommend that a geotechnical investigation be conducted at the site to confirm the subsurface condition and low seismic ground deformation hazards. Additionally, the seismic performance of the reservoir structure will need to be evaluated to ensure seismic resiliency under strong seismic ground shaking.

- **Townsend Road Tank** – This facility is a 0.015 MG, steel tank, constructed in 2009. This tank is located in the hills to the southwest of the City. There is no existing geotechnical information available for review. Although there are steep slopes nearby, the geologic map and nearby well logs indicate shallow rock in the area. We consider the seismic hazards in terms of liquefaction, lateral spreading, and landslide to be low at this site. We recommend that a geotechnical investigation be conducted at the site to confirm the subsurface condition and low seismic ground deformation hazards. Additionally, the seismic performance of the reservoir structure will need to be evaluated to ensure seismic resiliency under strong seismic ground shaking.

- **Fox Creek Reservoir, Dam, and Raw Water Pipeline** – This facility is an old (construction date unknown), 14.5-acre feet, 25 feet tall, wooden dam reservoir. The Fox Creek Reservoir is located in the hills to the southeast of the City. The reservoir dam site was not visited due to the access road blockage by fallen trees.

    - The reservoir's dam is constructed of wood and maintenance was recommended by the state inspector after the most recent inspection (Dam Safety Inspection Form and Report, dated January 28, 2021). There is no existing geotechnical information and as built records available for review. However, we understand that the reservoir dam is

surrounded by steep slopes nearby, and the geologic map and nearby well logs indicate the presence of landslide debris around the reservoir. Seismic settlement, lateral spreading and landslides may all be a hazard at this site.

- The Fox Creek raw water pipeline was recently reconstructed (date unknown). Available GIS data shows the pipeline running primarily on the eastern bank of Fox Creek. However, during the site visit the City representative informed us that this alignment was recently relocated and now runs under the (Unnamed?) access road, primarily on the west side of the creek. Geolocated data of the exact route was not available, however, either route passes through a steep banked canyon with several mapped landslide hazards on the order of 10 ft PGD. There is no existing geotechnical information available for review. We consider the seismic landslide hazard to be high in area.

- Considering the seismic and landslide vulnerability at the dam site and along the raw water pipeline, mitigation of geological hazards and improve the existing dam structure will be a very difficult and costly undertaken. We recommend to evaluate the need to include the Fox Creek reservoir and raw water pipeline as part of the seismic backbone system.

6.0 Conclusions and Recommendations

The City of Rainier's backbone water system extends across a wide range of seismic hazards. The major seismic hazard concerns are the high liquefaction and lateral spreading potentials in the Alluvial Sediment zone, adjacent to the Columbia River and in some localized drainage areas. In other areas, the estimated pipe fragility rate is generally low for the backbone pipes. However, the pipe system is primarily composed of CI pipes, which are considered to have higher seismic vulnerabilities in comparison to the other materials used in the backbone system.

Additionally, seismic evaluation and improvement to the Columbia River Intake structure and pipeline are recommended to increase the seismic resiliency of the system.

Furthermore, because of the lack of design and construction information on the existing facilities, we recommend conducting site-specific geotechnical and structural evaluations for each critical facilities along the backbone system (intake, water treatment plant and reservoirs) to ensure seismic resiliency.

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Principal Engineer

Devin Roth, E.I.
Sr. Staff Engineer

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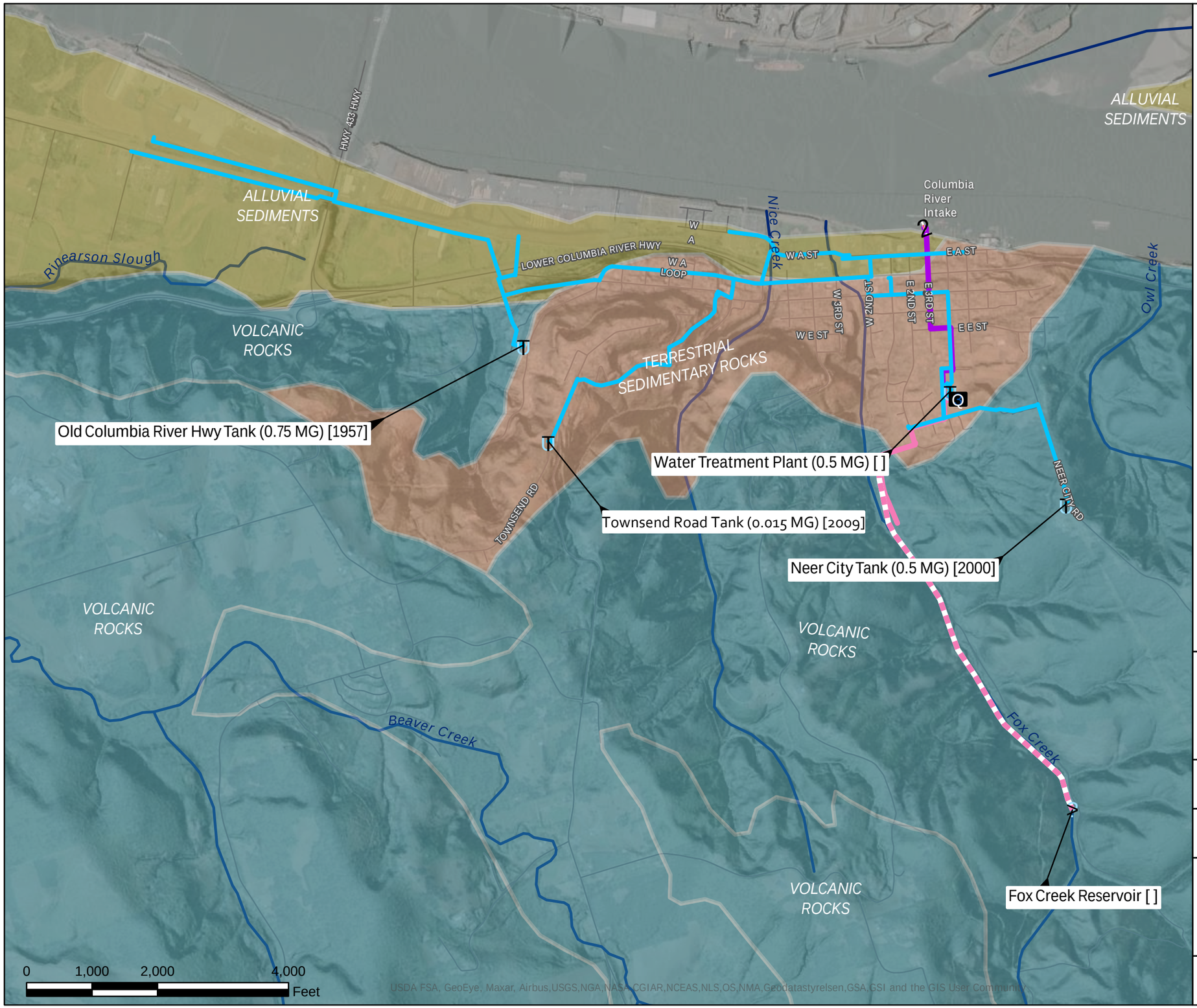
Figures



Legend

Rainier Water System

- Rainier Reservoir
- Existing Tank
- Intake
- Water Treatment Plant
- Columbia River Pipeline
- Fox Creek Pipeline
- Fox Creek, Unknown Alignment
- Critical Pipeline Backbone

Geology

- Alluvial Sediments
- Terrestrial Sedimentary Rocks
- Volcanic Rocks

NOTES:
SCALE: 1:18,000

DATA SHOWN ARE BASED ON DATA FROM DOGAMI Open File Report O-20-01 and ODGC7 AND HAVE NOT BEEN VERIFIED.

RAINIER WATER SYSTEM
SEISMIC RESILIENCE EVALUATION

GEOLOGIC MAP

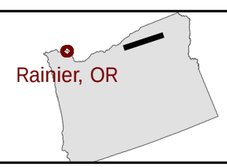


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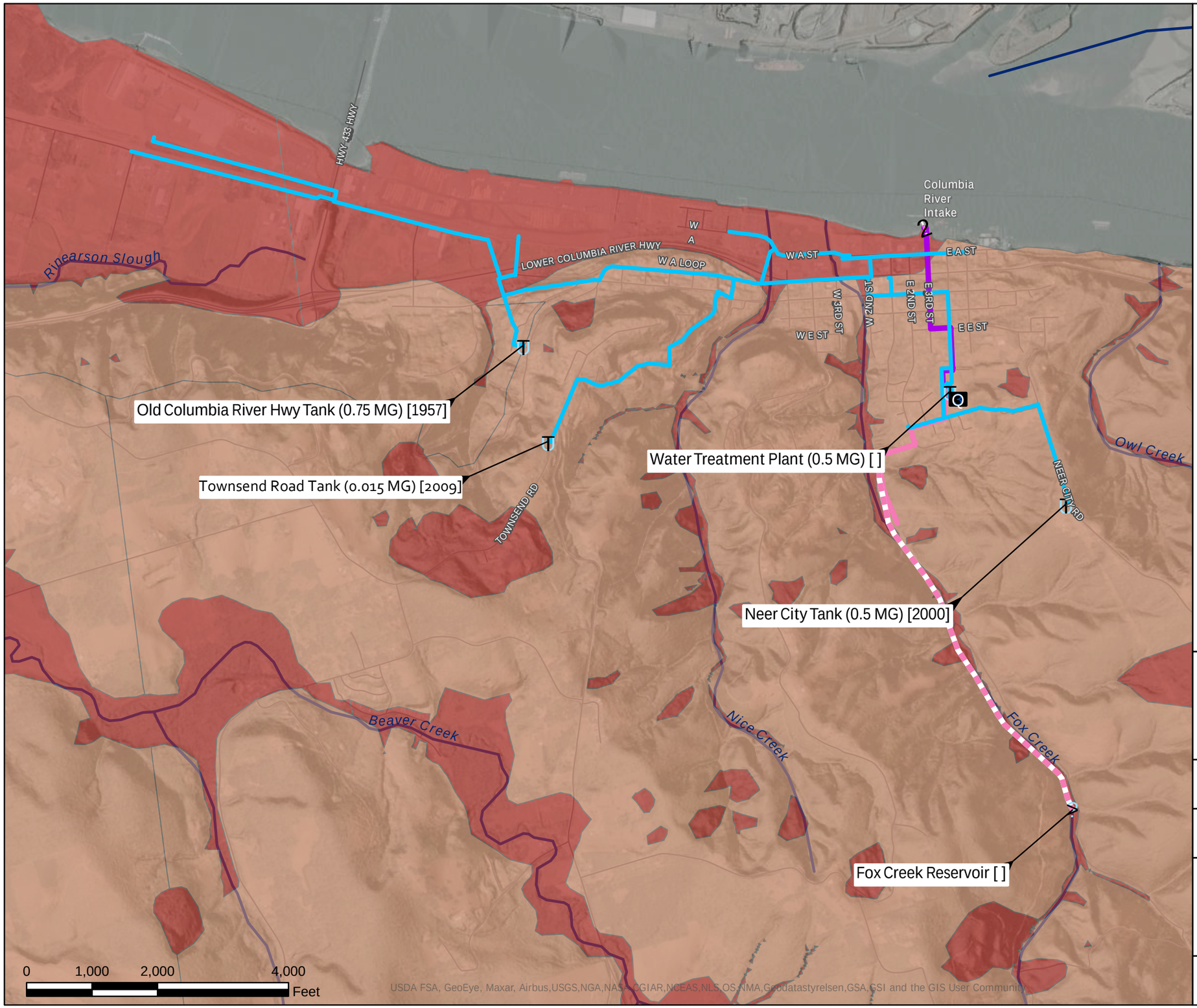
FIGURE 1



Legend

Rainier Water System

- Rainier Reservoir
- Existing Tank
- Intake
- Water Treatment Plant
- Columbia River Pipeline
- Fox Creek Pipeline
- Fox Creek, Unknown Alignment
- Critical Pipeline Backbone

Peak Ground Velocity

- 7 - 11 in/sec
- Up To 21 in/sec

NOTES:
SCALE: 1:18,000

DATA SHOWN ARE BASED ON DATA FROM DOGAMI Open File Report O-20-01 and ODGC7 AND HAVE NOT BEEN VERIFIED.

RAINIER WATER SYSTEM
SEISMIC RESILIENCE EVALUATION

PEAK GROUND VELOCITY MAP

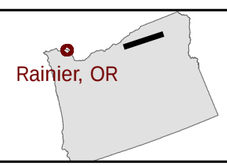


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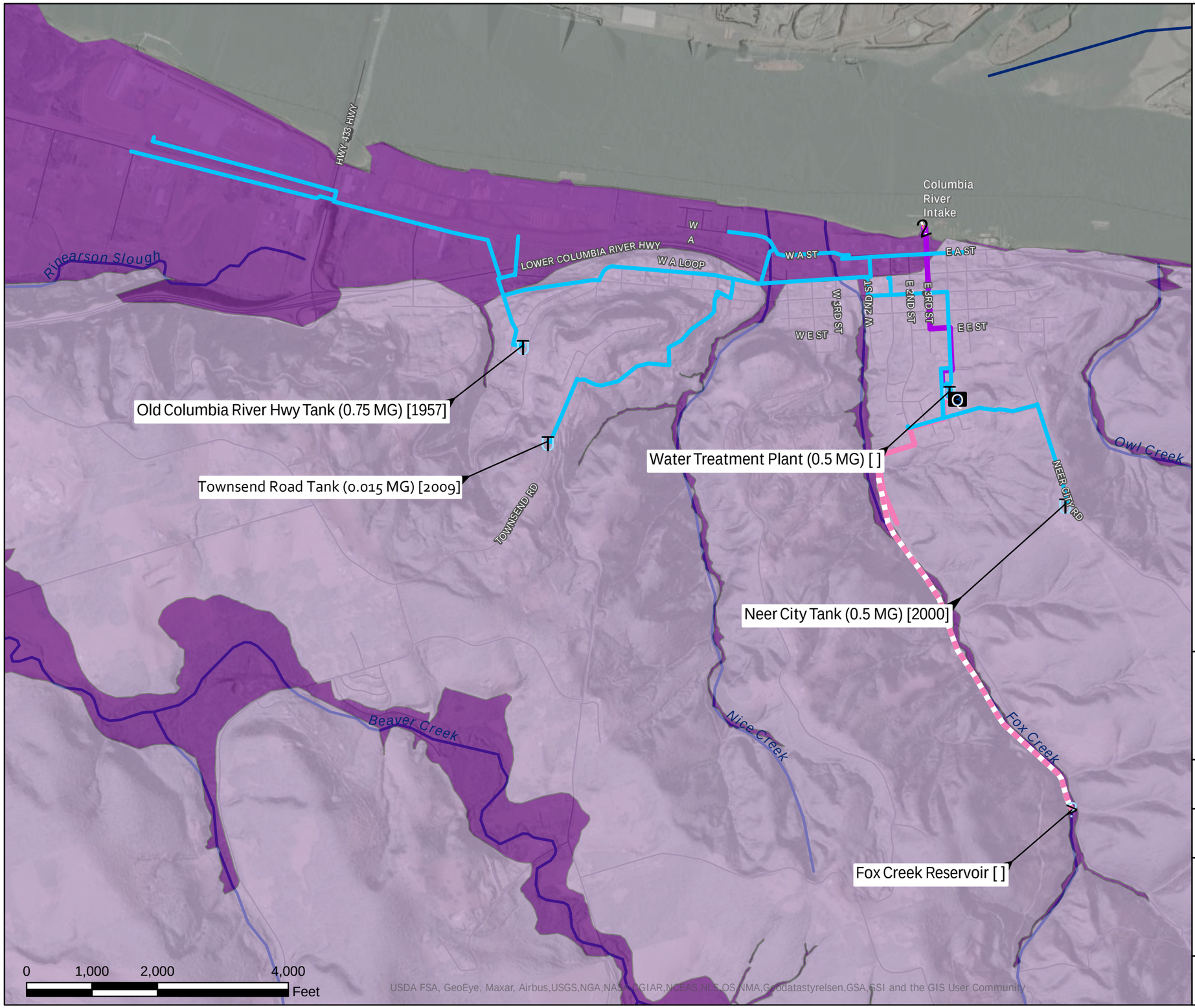
FIGURE 2



Legend

Rainier Water System

- Rainier Reservoir
- Existing Tank
- Intake
- Water Treatment Plant
- Columbia River Pipeline
- Fox Creek Pipeline
- Fox Creek, Unknown Alignment
- Critical Pipeline Backbone

Seismic Liquefaction Settlement

- None
- High (Up to 12 inches)

NOTES:
SCALE: 1:18,000

DATA SHOWN ARE BASED ON DATA FROM DOGAMI Open File Report O-20-01 and ODGC7 AND HAVE NOT BEEN VERIFIED.

RAINIER WATER SYSTEM
SEISMIC RESILIENCE EVALUATION

LIQUEFACTION SETTLEMENT MAP

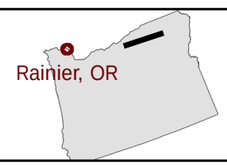


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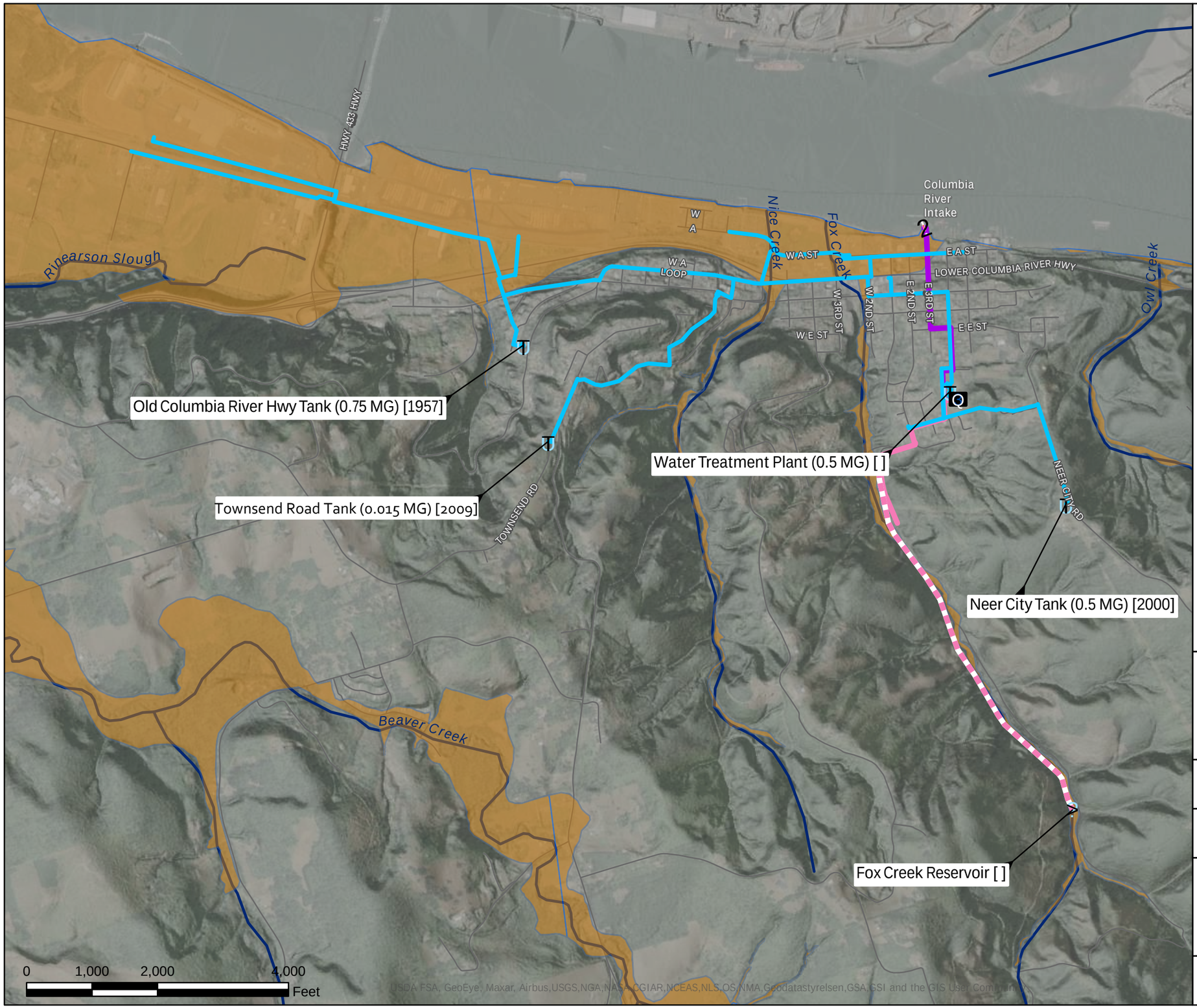
FIGURE 3



Legend

Rainier Water System

- Rainier Reservoir
- Existing Tank
- Intake
- Water Treatment Plant
- Columbia River Pipeline
- Fox Creek Pipeline
- Fox Creek, Unknown Alignment
- Critical Pipeline Backbone

Seismic Lateral Spread PGD

- None
- 15 feet

NOTES:
SCALE: 1:18,000

DATA SHOWN ARE BASED ON DATA FROM DOGAMI Open File Report O-20-01 and ODGC7 AND HAVE NOT BEEN VERIFIED.

RAINIER WATER SYSTEM
SEISMIC RESILIENCE EVALUATION

LIQUEFACTION LATERAL SPREADING MAP

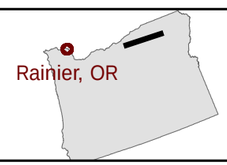


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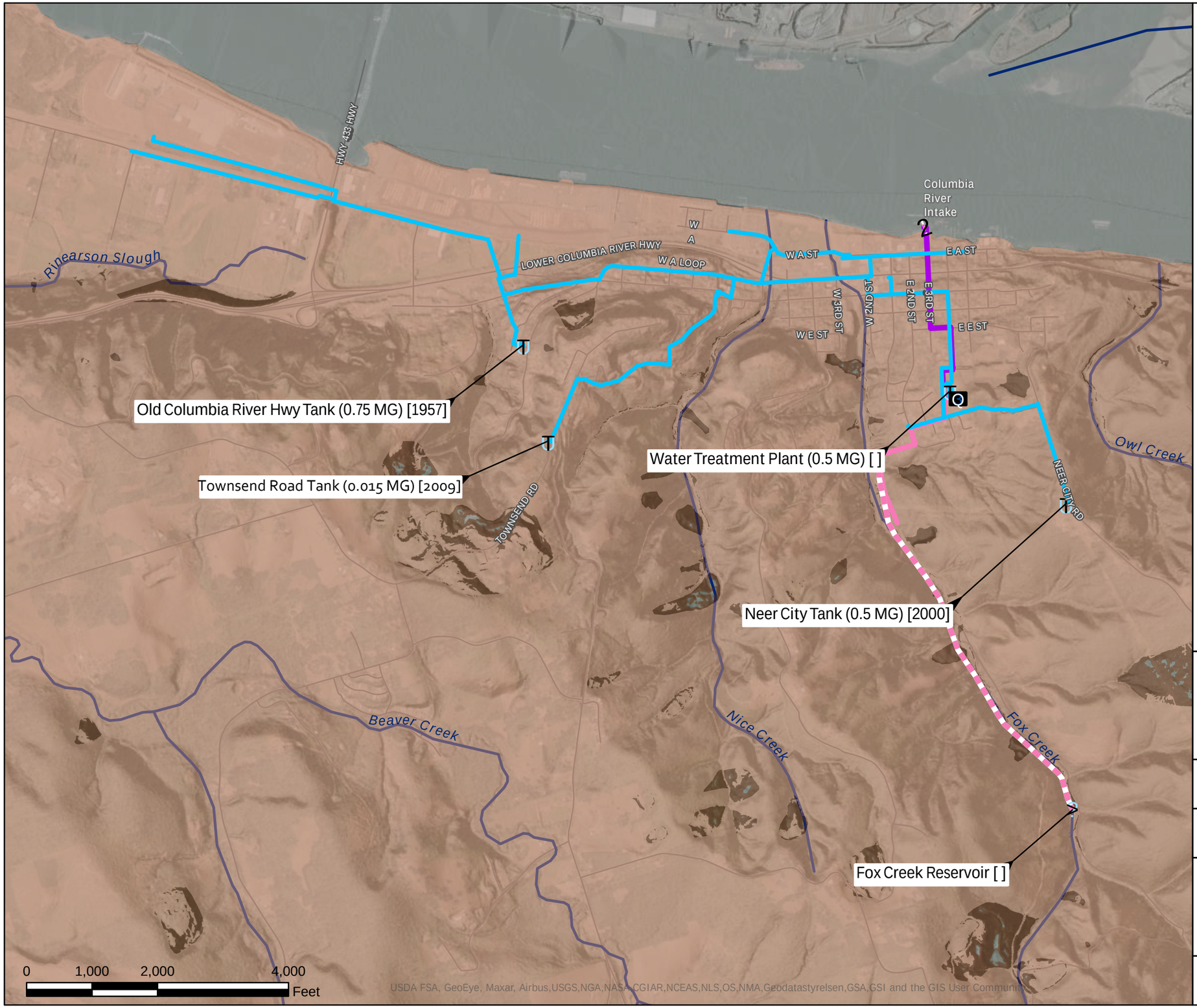
FIGURE 4



Legend

Rainier Water System

- Rainier Reservoir
- Existing Tank
- Intake
- Water Treatment Plant
- Columbia River Pipeline
- Fox Creek Pipeline
- Fox Creek, Unknown Alignment
- Critical Pipeline Backbone

Seismic Landslide PGD

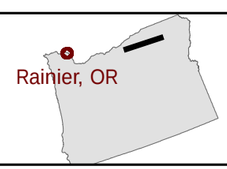
- Low
- High (10 Feet)

NOTES:
SCALE: 1:18,000

DATA SHOWN ARE BASED ON DATA FROM DOGAMI Open File Report O-20-01 and ODGC7 AND HAVE NOT BEEN VERIFIED.

RAINIER WATER SYSTEM
SEISMIC RESILIENCE EVALUATION

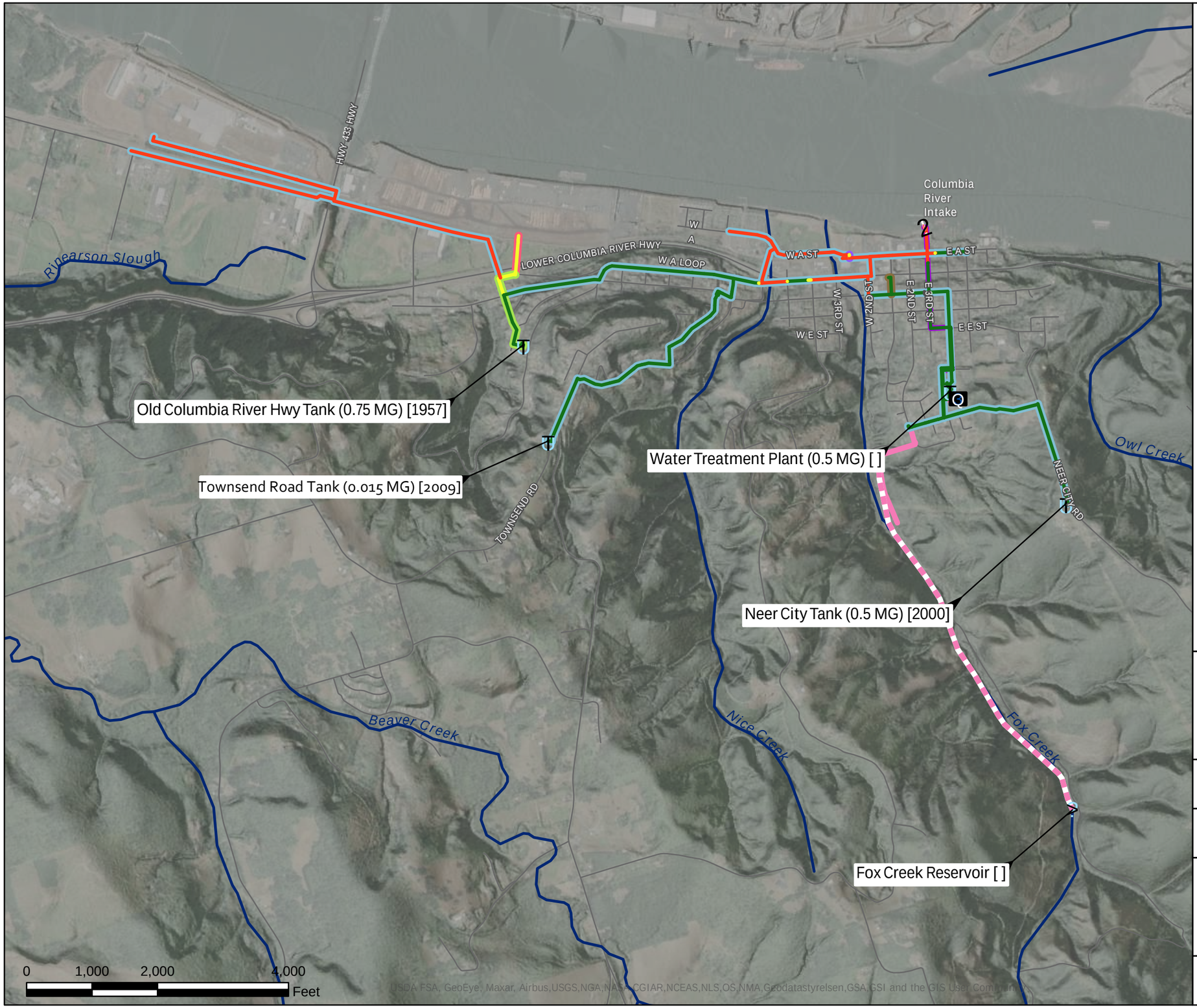
SEISMIC LANDSLIDE MAP



McMILLEN JACOBS ASSOCIATES

OCT 2022

FIGURE 5



Legend

Rainier Water System

- Rainier Reservoir
- Existing Tank
- Intake
- Water Treatment Plant
- Columbia River Pipeline
- Fox Creek Pipeline
- Fox Creek, Unknown Alignment
- Critical Pipeline Backbone

Material

- Cast Iron (<=12")
- Ductile Iron (<=12")
- Galvanized (<=12")
- Steel (16")
- Unknown (<=12")

Fragility Backbone Repair Rate

RR PGD ALA Repair Rate

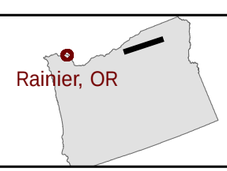
- < 1 (breaks/1000 feet)
- 1 - 4 (breaks/1000 feet)
- 4 - 6 (breaks/1000 feet)

NOTES:
SCALE: 1:18,000

DATA SHOWN ARE BASED ON DATA FROM DOGAMI Open File Report O-20-01 and ODGC7 AND HAVE NOT BEEN VERIFIED.

RAINIER WATER SYSTEM
SEISMIC RESILIENCE EVALUATION

PIPELINE FRAGILITY MAP



McMILLEN JACOBS ASSOCIATES

OCT 2022

FIGURE 6

Appendix A

Site Visit Photos

Photo 1: Columbia River Intake

Photo 2: Columbia River Intake Pump Station

Photo 3: Water Treatment Plant 0.5 MG Reservoir

Photo 4: Water Treatment Plant Building

Photo 5: Old Columbia River Hwy Tank (0.75 MG)

Photo 6: Groundwater Seepage and Slumping of Slope East of Old Columbia River Hwy Tank

Photo 7: Neer City Tank (0.5 MG)

Photo 8: Townsend Road Tank (0.015 MG)

DELVE underground

Technical Memorandum			
To:	Sandrine Ganry, PE Preston Van Meter, PE West Yost Associates, Inc.	Project:	City of Rainier Water Master Plan Phase 2
From:	Wolfe Lang, PE, GE Jeff Quinn, PE	cc:	File
Prepared by:	Wolfe Lang, PE, GE Jeff Quinn, PE	Job No.:	6401.0
Date:	April 27, 2023		
Subject:	Seismic Hazards Evaluation		

Revision Log

Revision No.	Date	Revision Description
0	December 13, 2022	Draft Submittal
1	April 27, 2023	Final Submittal

1.0 Introduction

The City of Rainier (City) contracted West Yost Associates, Inc. (West Yost) to provide professional services for the master plan updates and resilience study. As part of the study, a seismic resiliency evaluation of the water system is required in conjunction with the Oregon Resilience Plan (OSSPAC, 2013). West Yost retained Delve Underground to conduct a seismic hazard evaluation and backbone pipe vulnerability assessment; the results of which were presented in the technical memorandum City of Rainier Water Master Plan Update – Seismic Hazards Evaluation dated March 2021 (Delve Underground, formerly McMillen Jacobs, 2021).

The results of Delve Underground's seismic hazard evaluation indicated the following:

1. At the Columbia River Intake facility, seismic hazards and the risk of permanent ground deformation (PGD) due to soil liquefaction and lateral spreading are very high;
2. At the water treatment plant (WTP), seismic hazards and the risk of PGD due to soil liquefaction, lateral spreading, and seismically induced landsliding are low; and

3. Pipe damage due to ground shaking will be low. However, pipes in areas of large PGD, mainly due to liquefaction-generated lateral spreading near Columbia River or drainages, will suffer more damage.

The above conclusions are based on a desktop study of available geologic hazards mapping (DOGAMI, 2020) and nearby well logs (OWRD, 2022). Delve Underground recommended that a site-specific geotechnical investigation be conducted at the Columbia River Intake facility and the WTP to further assess the seismic hazards.

2.0 Purpose and Scope of Work

The purpose of our work for Phase 2 of the Project is to evaluate the seismic hazards (i.e., liquefaction, lateral spread, and seismic slope stability susceptibility) at the Columbia River Intake facility and the WTP based on site-specific subsurface data and provide mitigation or improvement recommendations to achieve the City's seismic resilience goals.

3.0 Site-Specific Geotechnical Investigation

The purpose of our work for Phase 2 of the Project is to evaluate the seismic hazards (i.e., liquefaction, lateral spread, and seismic slope stability susceptibility) at the Columbia River Intake facility and the WTP based on site-specific subsurface data and provide mitigation or improvement recommendations to achieve the City's seismic resilience goals.

3.1 Exploratory Borings

Delve Underground completed the site-specific geotechnical investigation for the Project on September 30, 2022. The investigation consisted of three exploratory borings (designated B-1 through B-3) completed by PLI Systems, Inc. of Hillsboro, Oregon, using a truck mounted B58 drill rig. Boring B-1, advanced near the Columbia River Intake pump station was advanced to a depth of 70 feet below ground surface (bgs). Borings B-2 and B-3 were advanced at the WTP site to respective depths of 30 to 45 feet bgs. Boring B-1 was advanced using mud-rotary drilling methods while borings B-2 and B-3 were advanced using hollow-stem auger drilling methods. The approximate boring locations are shown on the attached Site Plan (Figure 2). Summary boring logs are included in Appendix A.

The borings were completed under the supervision of a Delve Underground geologist who maintained continuous observation, collected soil, and maintained a full-depth descriptive log of the soil materials penetrated in each borehole.

3.2 Soil Classification & Sampling

Soil samples were classified in accordance with the Visual-Manual Procedure (ASTM D2488). Sample depths, stratigraphy, groundwater observations, and soil engineering characteristics

were also noted. The stratigraphic contacts indicated on the boring logs in Appendix A represent the approximate boundaries between soil types; actual transitions between soil units might be more gradational than shown.

Disturbed soil samples were collected at selected intervals using a standard 2-inch diameter split-barrel sampler and automatic safety hammer system. In each test, the sampler was advanced 18 inches by dropping a 140-pound hammer 30-inches for each strike in accordance with ASTM D1586. The number of hammer-blows for each 6 inches of penetration was recorded. The standard penetration resistance (designated as the "N-value") of the soil is calculated as the sum of the number of blows required for the final 12 inches of sampler penetration. SPT N-values of 50 or more blows per 6 inches or less of penetration is defined as "refusal." The N-value is an indication of the relative density of granular soils and the relative consistency of cohesive soils. The drill rig was equipped with an automatic hammer to obtain Standard Penetration Test (SPT) samples. PLI Systems, Inc. provided an automatic hammer calibration Report of SPT Hammer Energies (NV5, 2022), showing that the drill rig used for our site investigation (Mobile B58) has an energy transfer ratio of 82.6 (correction factor = 1.377). N-values reported on our boring logs are, however, uncorrected field-recorded values (i.e., no corrections have been applied).

3.3 Laboratory Testing

Soil samples were delivered to Delve Underground's Portland office, where each sample was re-examined and compared to the field boring log description to confirm the field classifications and maintain consistency. Representative samples were then selected for the following laboratory testing:

- Water (Moisture) Content of Soil and Rock by Mass (ASTM D2216);
- Amount of Material Finer than a No. 200 Sieve (i.e., 'Fines Content') (ASTM D1140); and
- Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils (ASTM D4318).

Laboratory testing was performed by Breccia Geotechnical Testing, LLC., of Tigard, Oregon, in accordance with the above-referenced ASTM standards. Laboratory tests were used to characterize soil properties and refine soil classifications. Laboratory test results are presented in Appendix B.

4.0 Site Conditions

4.1 Surface Conditions

4.1.1 Columbia River Intake Facility

The Columbia River intake facility is on the south banks of the Columbia River, at the intersection of East A and East Third Streets. The intake facility consists of an approximately 15- by 15-foot, single-story, reinforced concrete masonry pump house that sits over an 8-foot inside diameter, 32-foot deep wet well. Raw water from the Columbia River is conveyed to the wet well via a 12-inch diameter intake pipe that extends about 150 feet offshore, the end of which is adjacent to the boat dock and protected by a group of timber pilings. The pump station building is located about 100 feet south of the Columbia River shoreline, at the northeast corner of the City of Rainier Marina parking lot and at the top of an approximately 12-foot-tall fill slope. The fill slope descends at approximately 2H:1V (horizontal to vertical) to the north (i.e., towards the river) and to the east to the sandy beach along the Columbia River.

From the Columbia River intake facility, topography rises very gently (e.g., about 1 percent) to the south from approximately elevation (El.) 25 feet at the pump station to El. 30 feet at East A Street. From East A Street, the topography rises more sharply to the south from approximate El. 30 feet to El. 70 feet on the south side of Highway 30. From the south side of Highway 30, topography rises more steeply to the south, into the hills south of downtown and towards the WTP site.

4.1.2 Water Treatment Plant

The City of Rainier WTP is situated in the hills south of Rainier at 750 Rainier Boulevard. The WTP includes a 0.5MG storage tank, a 0.25MG backwash lagoon basin, an approximate 45- by 50-foot storage building, and the approximate 50- by 60-foot main WTP building. Access to the WTP is provided by a fenced gate on the west side from Rainier Boulevard. An access road runs approximately southeast-northwest through the WTP site. The WTP site can be divided into two approximate halves: the lower/northern half and the upper/southern half.

The lower/northern half of the WTP site sits between approximate El. 239 and 243 feet and contains the storage building and the storage tank. Most of the area between the storage tank and the storage building is asphalt paved. To facilitate the construction of the storage tank, an approximately 10- to 15-foot-tall cut slope was made into the hillside on the south and southeast sides of the tank. The cut slope is inclined at approximately 1.5H:1V and is concrete-lined. From the crest of the cut slope, topography rises to the south at approximately 5H:1V before transitioning to the upper/southern half of the site.

The upper/southern half of the WTP site sits between approximate El. 265 and 269 feet and contains the main WTP building and the stilling basin. To facilitate the construction of the main WTP building, a cut slope was constructed on the east side of the building, as evidenced by an

approximately 3-foot-tall retaining wall around the eastern and southern sides of the building. From the east side of the main WTP building, topography rises from approximate El. 269 feet to El. 329 (i.e., near the top of the hill), at slope gradients ranging from about 4H:1V to 2H:1V. The stilling basin is located approximately 25 feet from the southwest corner of the main WTP building.

4.2 Subsurface Conditions

Detailed discussions regarding regional geology and geologic units encountered in Rainier, Oregon were provided in the previous technical memorandum (Delve Underground, 2021). The following sections provide details regarding the subsurface conditions, and how they relate to mapped geologic conditions, encountered during our site-specific geotechnical investigation at the Columbia River intake facility and the WTP site.

4.2.1 Columbia River Intake Facility

Boring B-1 was advanced about 35 feet southwest of the pump house in the parking lot. Below the pavement section (3 inches of asphalt over 18 inches of base aggregate), subsurface conditions consist of the following:

- *Artificial Fill*: Artificial fill consisting of poorly graded sand (SP) was encountered below the pavement section to about 12 feet bgs and likely sourced from dredged river materials. SPT N-values ranged from 14 to 29 blows per foot (bpf), indicating medium dense conditions. Geologic mapping (Gless, 1989) indicates fill materials are present along the waterfront and generally extend to Highway 30.
- *Holocene Alluvium*: Below the fill, silty sand (SM) alluvium sediments extend to about 33 feet bgs. Fines content of the silty sand ranges from 27 to 40 percent based on two fines content tests. The silty sand contains occasional, relatively clean sand lenses. SPT N-values in this unit range from 1 to 14 bpf, indicating very loose to medium dense conditions.
- *Residual Soil of Goble Volcanics*: Residual soils derived from deep weathering of volcaniclastic sedimentary and volcanic rocks extend to the terminal depth of 70 feet. These soils exhibit high plasticity based on one Atterberg Limits test showing a Liquid Limit (LL) of 66 and a Plasticity Index (PI) of 37. The residual soils have a heterogeneous structure indicative of a mixture of marine sedimentary and volcanic rocks. SPT N-values range from 6 to 23 bpf (increasing with depth), indicating medium stiff to very stiff conditions. The residual soils are consistent with the Goble Volcanics (geologic map unit: Tgo1) in published geologic mapping (Phillips, 1987).

4.2.2 Water Treatment Plant Site

Subsurface conditions at the WTP site consist of variable depths of colluvium, derived from the residual soil of the Goble Volcanics that moved down-slope over time due to gravitational

forces. The colluvium is underlain by alluvial terrace deposits, as the WTP site is located on one of several stream terraces on the hillsides of Rainier, resulting from the Missoula Floods which affected elevations up to about 300 feet at Rainier (Gless, 1989). The terrace deposits are underlain by volcaniclastic sedimentary rocks of the Goble Volcanics (Phillips, 1987). Further details of subsurface conditions encountered at the WTP site are discussed in the following sections.

4.2.2.1 Upper Area – Main WTP Building

Boring B-2 was advanced about 20 feet north of the main WTP building in the pavement. Below the pavement section (2 inches of asphalt over 12 inches of base aggregate), subsurface conditions consist of the following:

- *Colluvium*: Below the pavement, colluvium was encountered to about 10 feet bgs. SPT N-values ranged from 5 to 11 blows per foot (bpf). The colluvium consists of medium stiff to stiff, moist, orange-brown mottled, lean clay (CL) with an overall heterogeneous structure, exhibits low to medium plasticity, and contains trace fine gravel, fine sand, and charcoal/organics.
- *Alluvial Terrace Deposits*: The colluvium is underlain by alluvial terrace deposits extending to the terminal depth explored of 30 feet. These soils consist of medium stiff to very stiff, moist, orange with brown mottling, sandy silt (ML) that exhibits low plasticity. Fines content ranges from 55 to 64 percent based on two fines content tests. SPT N-values range from 6 to 19 bpf.

4.2.2.2 Lower Area – Storage Tank and Storage Building

Boring B-3 was advanced about 40 feet north of the storage tank in a paved area. Below the pavement section (5 inches of asphalt over 12 inches of base aggregate), subsurface conditions consist of the following:

- *Colluvium*: Below the pavement, colluvium was encountered to about 18 feet bgs. The colluvium consists of stiff, moist, orange-brown mottled and light gray with black and orange mottling, fat clay (CH) with an overall heterogeneous structure exhibiting high plasticity. SPT N-values ranged from 9 to 13 blows per foot (bpf).
- *Alluvial Terrace Deposits*: The colluvium is underlain by alluvial terrace deposits extending to approximately depth of 40.5 feet bgs. The terrace deposits consist of medium dense, moist, orange with brown mottling, silty sand (SM). Fines content ranges from 18 to 38 percent based on two fines content tests. SPT N-values range from 17 to 28 bpf.
- *Residual Soil of Goble Volcanics*: Below the alluvial terrace deposits, high plasticity, clay-rich residual soils were encountered and extend to the terminal depth explored of 45 feet. The residual soils exhibit high plasticity behavior and have an overall heterogeneous structure that is typically indicative of a mixture of marine sedimentary

and volcanic rocks. SPT N-values in this unit range from 15 to 42 bpf, indicating stiff to hard conditions.

4.3 Groundwater

4.3.1 Columbia River Intake Facility

Boring B-1 was advanced using mud rotary drilling methods, which precludes the observation of groundwater conditions during drilling. However, we observed saturated soils at 13 feet bgs; which corresponds approximately to the elevation difference between the water surface level in the Columbia River and the ground surface. We anticipate groundwater levels at the pump house will be closely tied to water surface levels in the adjacent Columbia River.

For our analyses, we used a groundwater level of 10 feet bgs. This is consistent with groundwater levels measured in a geotechnical boring located about 100 feet southeast of pump station building based on our Oregon Water Resources Department (OWRD) online well log query (OWRD, 2022).

4.3.2 Water Treatment Plant Site

Borings B-2 and B-3 were advanced using hollow stem auger drilling methods, which allow for the observation and measurement of groundwater. We did not observe groundwater while advancing these borings. However, there is potential for perched groundwater conditions during heavy rainfall within the silty sand and sandy silt alluvial terrace deposits overlying relatively impermeable, clay-rich residual soil that acts as an aquitard to the downward seepage of water. We anticipate that groundwater and perched groundwater levels will fluctuate due to seasonal and annual variations in precipitation, changes in site utilization, or other factors.

5.0 Seismic Hazards

5.1 Seismic Parameters of CSZ Event

The Oregon Department of Geology and Mineral Industries (DOGAMI) published a series of seismic maps for a simulated M9.0 Cascadia Earthquake (DOGAMI, 2013) to identify potential seismic hazards for the development of the 2013 Oregon Resilience Plan (OSSPAC, 2013), including peak ground acceleration (PGA) and peak ground velocity (PGV). Other parameters used for the geotechnical analyses for our seismic hazards evaluation include earthquake magnitude (M), mean site-to-source distance (R), and groundwater levels.

Table 1. Seismic Parameters for Geotechnical Analysis

Seismic Parameter	Facility/Location	
	Columbia River Intake Facility	Water Treatment Plant
PGA	0.35g	0.22g
PGV	20 inches/second	10 inches/second
Earthquake Magnitude, M	9.0	9.0
Mean Site-to-Source Distance[1], R	83.1 km	83.2 km
Groundwater Level	10 feet bgs[2]	n/a[3]

Table Notes:

1. Mean site-to-source distance obtained from USGS deaggregation data for the interface seismic source type associated with a M9.0 CSZ earthquake.
2. Groundwater level is based on observations made during wet winter months in geotechnical boring located about 100 feet southeast of pump station building (i.e., conservative groundwater level).
3. Groundwater was not observed within the maximum depths explored at the WTP site.

5.2 Liquefaction

Liquefaction occurs when ground shaking from an earthquake transforms soil from a solid state to a viscous fluid state. Liquefiable soils are typically saturated sands and non-plastic to low-plasticity silts below groundwater. Silts and silty soils with a plasticity index less than 7 are generally considered to be liquefiable. Consequences of liquefaction include loss of shear strength, loss of soil materials through sand boils or flow, flotation of buried chambers/pipes, and post-liquefaction reconsolidation (settlement).

We assessed liquefaction potential at the Columbia River Intake facility and the WTP site using liquefaction triggering analyses in accordance with Boulanger and Idriss (2014), which is a stress-based approach comparing seismically induced cyclic stresses to the cyclic resistance of the soil. The seismic stresses, or the cyclic stress ratio (CSR), is dependent on PGA, earthquake magnitude, and general stiffness of the soil. Resistance to cyclic failure, or the cyclic resistance ratio (CRR), is dependent on SPT N-Values and other soil properties. The ratio of the CSR to CRR is the factor of safety against triggering liquefaction. SPT N-values from the site-specific borings, along with the parameters summarized in Table 1, were used in our liquefaction triggering analysis.

Our liquefaction evaluation also included the calculation of liquefaction-induced settlement for potentially liquefiable soils (i.e., a factor of safety of less than 1 from the triggering analysis). We evaluated vertical, post liquefaction induced settlements using methods proposed by Ishihara and Yoshimine (1992), following the procedures presented in Idriss and Boulanger (2008).

The results of our liquefaction analysis are summarized as follows:

- *Columbia River Intake Facility*: Liquefiable soils are present between 10 feet (i.e., design groundwater level) and 33 feet bgs. The liquefiable soils include the lower portion of the clean dredge sand and the alluvial sediments unit, which typically consists of very loose to loose silty sand. Vertical, post-liquefaction settlement is on the order of 8 inches, which is relatively consistent with published DOGAMI liquefaction settlement mapping (DOGAMI, 2020) that shows up to 12 inches of liquefaction settlement along the Columbia River.
- *Water Treatment Plant*: Due to the lack of groundwater and saturated soils observed during our geotechnical exploration, as well as the relatively stiff/dense soil conditions, we conclude the risk of liquefaction at the WTP site is low. This conclusion is consistent with DOGAMI liquefaction settlement mapping (DOGAMI, 2020), which shows zero post-liquefaction settlement.

5.3 Lateral Spreading

Lateral spreading is a liquefaction-related phenomenon resulting in ground displacement during an earthquake and occurs in sloping ground or flat ground with a free face. The pump house is adjacent to a riverbank slope (e.g., a free face) and the underlying alluvial deposits are susceptible to post-liquefaction failure and are therefore susceptible to lateral spreading. Based on a recent boring advanced where Fox Creek crosses Highway 30 in Rainier, we encountered alluvial deposits to a depth of approximately 22 feet. Therefore, these soils extend at least 500 feet south of the pump house and likely pinch out further to the south as the hills climb in elevation. The ground surface generally descends at an approximate 4-percent grade from Highway 30 towards the Columbia River.

We evaluated lateral spreading using the Bardet et al. (2002) and Youd et al. (2002) analytical methods to estimate the resulting lateral displacement, as summarized in Table 2. We estimated slope geometry parameters by generating cross sections in Google Earth Pro. Both methods are multi-linear regression analyses developed using a database of liquefaction case histories and can be used for free-face and sloping ground conditions.

Table 2. Estimated Lateral Spreading Displacement

Conditions	Input Parameters[1]	Estimated Lateral Displacement (feet), (Bardet et al., 2002)	Estimated Lateral Displacement (feet), (Youd et al., 2002)
Free Face	Free-Face Height, H: 13 feet Free-Face Setback[2], L: 40 feet T_{15}[3]: 7 feet $D50_{15}$[4]: 0.15 mm F_{15}[5]: 35 percent $D50_{15}$[4] = 0.15 mm	5.0	10.9
Sloping Ground	Slope Gradient, S: 4 percent T_{15}[3]: 10 feet $D50_{15}$[4] = 0.15 mm F_{15}[5]: 50 percent $D50_{15}$[4] = 0.15 mm	4.8	7.6

Table Notes

1. In addition to the above parameters, an earthquake magnitude of 9.0, PGA of 0.35g, and site-to-source distance of 83 km were used in the lateral spreading analyses.
2. Free-face setback is measured from middle of the pump house to the base of the streambank/fill slope.
3. T_{15} is the cumulative thickness of saturated sandy layers with $(N_1)_{60}$ values of less than 15 blows per foot, used in both methods.
4. $D50_{15}$ is the average grain size of the layer(s) included in T_{15}, used in Youd et al., 2002 method only.
5. F_{15} is the average fines content of saturated granular layers included in T_{15}, used in Youd et al., 2002 method only.

As reported in our previous study (Delve Underground, 2021), we used historic well logs to assess the subsurface conditions and confirm that the magnitude of lateral spreading along the Columbia River on the DOGAMI lateral spreading maps estimates are reasonable. Based on our site-specific geotechnical investigation at the intake facility, we calculated lateral spread displacements ranging from about 5 to 11 feet, or about 33 to 75 percent of the mapped estimates.

The soils underlying the WTP site are not susceptible to liquefaction and therefore, we conclude that the risk of lateral spread at the WTP site is low.

5.4 Seismic Landslides

Earthquake induced landslides can occur on slopes due to the inertial force from an earthquake adding load to a slope. The ground movement due to landslides can be extremely large and

damaging to pipelines and other structures. At the WTP site, due to the stiff subsurface conditions and deep groundwater table, the risk of seismic landslides is estimated to be low.

6.0 Conclusions

The Columbia River Intake Facility is at high risk for significant damage to complete loss of serviceability after a CSZ earthquake. Permanent ground deformations at the intake facility include about 8 inches of post-liquefaction settlement and lateral spread ranging from about 5 to 11 feet. To mitigate the liquefaction and lateral spreading hazards at the intake facility, extensive ground improvement, such as jet grouting, will be required. This will be expensive (i.e., more than $1.0M) with potential environmental concerns due to grout leakage into the river. In addition, mitigation or replacement of the in-water intake pipe/screen, structural and mechanical improvements for the existing pump station/wet-well and on-land raw water pipe (extending beyond the lateral spreading zone) will be needed. These make the seismic improvement of existing intake very extensive and expensive. Alternatively, other intake solutions can be considered (see Section 7.0).

Based on the results of our site-specific geotechnical investigation and engineering analysis, the WTP site has a low risk of seismic hazards. Therefore, we consider the WTP to be seismically resilient from a geotechnical and geologic hazard perspective. However, we do recommend that a structural engineer evaluate the existing WTP structures for strong ground shaking during a long duration CSZ earthquake to identify any potential vulnerabilities.

7.0 Recommendations

Based on a recent meeting with West Yost and the City, we understand that the City is not interested in hardening the existing Columbia River Intake Pump Station and associated pipelines because the complexity and cost. We understand that there are two alternatives being considered regarding the mitigation or hardening of the City's raw water supply system.

1. Rebuild the raw water intake facility on a site that is not susceptible to liquefaction and lateral spreading hazards (i.e., on the east side of Rainier where shallow bedrock is anticipated).
2. Utilize Fox Creek Reservoir as an alternative raw water source while a new primary raw water intake facility is constructed (i.e., assuming the existing intake facility fails). However, Fox Creek Reservoir is impounded by an aging wood dam built in the early 1900s that is not likely seismically resilient itself. Therefore, the City may be interested in the potential rehabilitation of the existing dam.

We visited the Fox Creek Dam and Reservoir and met with the City on December 5th, 2022, to discuss a potential rehabilitation approach. The existing dam is a wooden structure supported on concrete grade-beam foundation. The dam is approximately 25 feet tall, 50 feet wide, and 200 feet long at the crest (about 75 to 100 feet long at the base). The volume of the reservoir is

about 1,400 acre-feet. The dam is classified as high hazard due to the proximity of downstream residences. The abutments appear to be stable and the dam appears to be founded on marine sedimentary rocks. The City also mentioned that there is a waterfall downstream from the dam and is therefore not likely a habitat for fish.

Considering the above, we think it is feasible to improve and rehabilitate the existing dam with the following approach:

- Drain the reservoir, then dredge and clean the bottom as necessary;
- Assess the existing grade-beam foundations and strengthen them with short ground anchors installed into bedrock;
- Remove and over-excavate, if necessary, the soils between grade-beams to competent bedrock and then replace with concrete;
- Replace the outlet pipe and gate;
- Fill the dam with concrete by using the existing wooden dam face as forms. Additional forms on the downstream face of the dam will be needed. Also, some localized supports or reinforcements may be required;
- Improve the spillway; and
- Install an impermeable liner at the upstream face, toe and at the abutments.

We estimated the cost for the dam improvement/rehabilitation will be on the order of $6.0M which will include design (geotechnical and stability analyses, hydraulic and dam breach analyses, permitting, engineering and design) and construction (access improvement, mobilization, and improvements). A detailed breakdown of the planning level cost estimate is included in Appendix C.

8.0 Closure

This report has been prepared for the exclusive use of the City of Rainier and West Yost Associates, Inc. for the City of Rainier Water Master Plan Phase 2 project. The data presented in this report is based on the subsurface conditions encountered during our site explorations. The data presented herein is intended to support the design of the proposed improvements. Delve Underground is not responsible for the interpretation of the data contained in this report by anyone; as such interpretations are dependent on each person's subjectivity.

In the performance of geotechnical work, specific information is obtained at specific locations at specific times, and geologic conditions can change over time. It should be acknowledged that variations in soil conditions may exist between exploration and exposed locations and this report does not necessarily reflect variations between different explorations. The nature and extent of variation may not become evident until construction. If, during construction,

conditions observed or encountered differ from those disclosed by this report, Delve Underground should be advised at once so we can observe and review these conditions and reconsider our recommendations where necessary.

The geotechnical engineering evaluations and interpretations are completed within the limitations of Delve Underground's approved scope of work, schedule and budget. The services rendered by Delve Underground have been performed in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions in the same area. The construction recommendations are considered preliminary and provided for planning purposes only. Delve Underground is not responsible for the use of this report in connection with anything other than the project at the location described above.

DELVE UNDERGROUND

Wolfe Lang, P.E., G.E.
Principal Geotechnical Engineer

Jeff Quinn, P.E.
Associate Engineer

9.0 References

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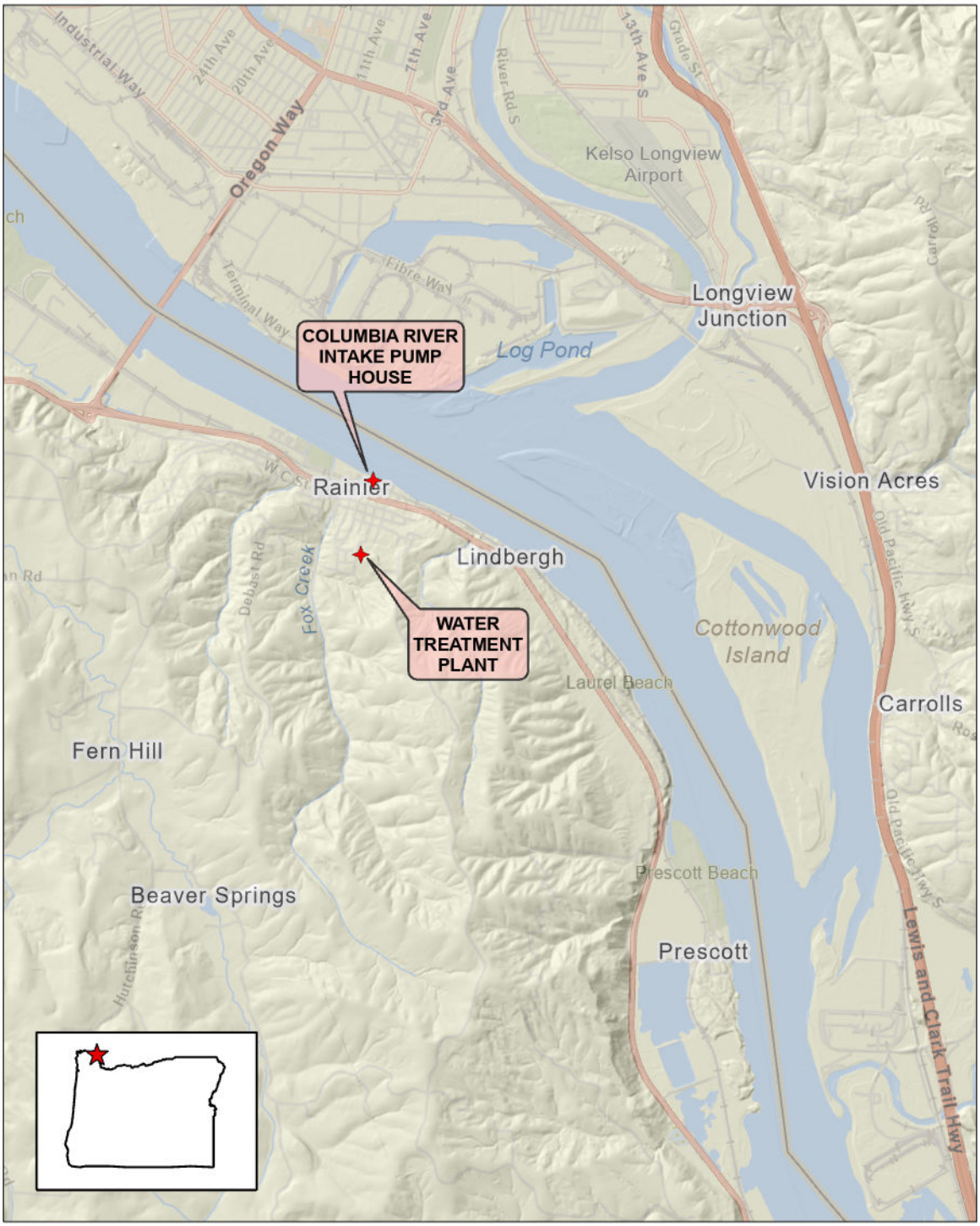
Figures



PROJECT VICINITY MAP

SCALE: 1 inch = 1 miles

DELVE underground

CITY OF RAINIER

WATER MASTER PLAN PHASE 2

GEOTECHNICAL ENGINEERING REPORT
VICINITY MAP

FIG.1

APRIL 2023

SITE PLAN - WATER TREATMENT PLANT

SCALE: 1 inch = 100 feet

DELVE underground

CITY OF RAINIER

WATER MASTER PLAN PHASE 2

GEOTECHNICAL ENGINEERING REPORT
SITE PLAN - WATER TREATMENT PLANT

FIG.2A

APRIL 2023

SITE PLAN - PUMP HOUSE

SCALE: 1 inch = 100 feet

DELVE underground

CITY OF RAINIER

WATER MASTER PLAN PHASE 2

GEOTECHNICAL ENGINEERING REPORT
SITE PLAN - PUMP HOUSE

FIG.2B

APRIL 2023

Appendix A

Boring Logs

MOISTURE CONTENT

DESCRIPTION	CONDITION
Dry	Absence of moisture, dusty, dry to the touch.
Moist	Damp, but no visible water.
Wet	Visible free water, typically below water table.

ABBREVIATIONS

SYMBOL	DEFINITION
⊢	Atterberg Limits
○	Moisture Content
□	Blows per foot (N)

FINE-GRAINED SOIL CONSISTENCY

RELATIVE CONSISTENCY	N, SPT *Blows/foot*
Very Soft	0 to 1
Soft	2 to 4
Medium stiff	5 to 8
Stiff	9 to 15
Very Stiff	16 to 30
Hard	> 30

COARSE-GRAINED SOIL DENSITY

Relative Density	N, SPT *Blows/foot*
Very Loose	0 to 4
Loose	5 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	> 50

PERCENTAGE RANGE TERMS[1,2]

DESCRIPTION	RANGE
Trace	< 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

1. Gravel, Sand and fines are estimated by mass. Other constituents such as organics, cobbles, and boulders are estimated by volume.
2. Percentages per ASTM D2488.

SOIL CONSTITUENCY DEFINITIONS

CONSTITUENT	COARSE-GRAINED	FINE-GRAINED
Major	Less than 50% fines: **SAND** or **GRAVEL**	More than 50% fines: **SILT, ELASTIC SILT, LEAN CLAY, FAT CLAY, ORGANIC SOIL**
Secondary	12%[1] or more fine-grained: **Silty** or **Clayey**	30% or more coarse-grained: **Sandy** or **Gravelly**
Minor	5 to 12%[1] fine-grained: **with Silt** or **with Clay**	15 to 30% coarse-grained: **with Sand** or **with Gravel**
	15% or more of a second coarse-grained constituent: **with Sand** or **with Gravel**	30% or more total coarse-grained and the lesser coarse constituent is 15% or more: **with Sand** or **with Gravel**

1. ASTM D2488 specifies more than 15% fines

PARTICLE SIZE DEFINITIONS

DESCRIPTON		SIEVE SIZE *PER ASTM D2488*
FINES		< #200 (0.075 mm)
SAND	*Fine*	#200 to #40 (0.075 to 0.4 mm)
	Medium	#40 to #10 (0.4 to 2 mm)
	Coarse	#10 to #4 (0.4 to 4.75 mm)
GRAVEL	*Fine*	#4 to ¾ in. (4.75 to 19 mm)
	Medium	¾ to 3 in. (19 to 76 mm)
COBBLES		3 to 12 in. (76 to 305 mm)
BOULDERS		> 12 in. (305 mm)

DELVE underground

KEY TO SUBSURFACE LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)[1]

MAJOR DIVISIONS			SYMBOL		TYPICAL DESCRIPTION	ALTERNATE DESCRIPTIONS
COARSE-GRAINED SOILS (50% OR MORE RETAINED BY NO. 200 SIEVE)	GRAVELS (MORE THAN 50% RETAINED ON NO. 4 SIEVE)	CLEAN GRAVELS (≤ 5% FINES)	GW		WELL-GRADED GRAVEL	WELL-GRADED GRAVEL WITH SAND
			GP		POORLY GRADED GRAVEL	POORLY GRADED GRAVEL WITH SAND
		GRAVELS[2,4] (5 – 12 % FINES)	GW-GM		WELL-GRADED GRAVEL WITH SILT	WELL-GRADED GRAVEL WITH SILT AND SAND
			GW-GC		WELL-GRADED GRAVEL WITH CLAY	WELL-GRADED GRAVEL WITH CLAY AND SAND
			GP-GM		POORLY GRADED GRAVEL WITH SILT	POORLY GRADED GRAVEL WITH SILT AND SAND
			GP-GC		POORLY GRADED GRAVEL WITH CLAY	POORLY GRADED GRAVEL WITH CLAY AND SAND
		GRAVELS WITH FINES[2] (≥ 12% FINES)	GM		SILTY GRAVEL	SILTY GRAVEL WITH SAND
			GC		CLAYEY GRAVEL	CLAYEY GRAVEL WITH SAND
	SANDS (LESS THAN 50% RETAINED ON NO. 4 SIEVE)	CLEAN SANDS (≤ 5% FINES)	SW		WELL-GRADED SAND	WELL-GRADED SAND WITH GRAVEL
			SP		POORLY GRADED SAND	POORLY GRADED SAND WITH GRAVEL
		SANDS[2,4] (5 – 12 % FINES)	SW-SM		WELL-GRADED SAND WITH SILT	WELL-GRADED SAND WITH SILT AND GRAVEL
			SW-SC		WELL-GRADED SAND WITH CLAY	WELL-GRADED SAND WITH CLAY AND GRAVEL
			SP-SM		POORLY GRADED SAND WITH SILT	POORLY GRADED SAND WITH SILT AND GRAVEL
			SP-SC		POORLY GRADED SAND WITH CLAY	POORLY GRADED SAND WITH CLAY AND GRAVEL
		SANDS WITH FINES[3] (> 12% FINES)	SM		SILTY SAND	SILTY SAND WITH GRAVEL
			SC		CLAYEY SAND	CLAYEY SAND WITH GRAVEL
FINE-GRAINED SOILS (50% OR MORE PASSES NO. 200 SIEVE)	SILTS AND CLAYS (LL < 50)	INORGANIC	ML		SILT	SILT WITH SAND OR GRAVEL; SANDY OR GRAVELLY SILT
			CL		LEAN CLAY	LEAN CLAY WITH SAND OR GRAVEL; SANDY OR GRAVELLY LEAN CLAY
		ORGANIC	OL		ORGANIC SOIL	ORGANIC SOIL WITH SAND OR GRAVEL; SANDY OR GRAVELLY ORGANIC SOIL
	SILTS AND CLAYS (LL ≥ 50)	INORGANIC	MH		ELASTIC SILT	ELASTIC SILT WITH SAND OR GRAVEL; SANDY OR GRAVELLY ELASTIC SILT
			CH		FAT CLAY	FAT CLAY WITH SAND OR GRAVEL; SANDY OR GRAVELLY FAT CLAY
		ORGANIC	OH		ORGANIC SOIL	ORGANIC SOIL WITH SAND OR GRAVEL; SANDY OR GRAVELLY ORGANIC SOIL
	SILT/CLAY[2]	INORGANIC	CL-ML		SILTY CLAY	SILTY CLAY WITH SAND OR GRAVEL; SANDY OR GRAVELLY SILTY CLAY
HIGHLY ORGANIC SOILS		ORGANIC	PT		PEAT	

NOTES:

1. The USCS described here is based on ASTM standards D2487 & D2488.
2. Dual symbol materials (e.g., SP-SM) are used for soils between 5% and 12% fines or when liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart, (LL: 12 -25, PI: 4-7).
3. ASTM D2488 specifies the use of dual symbol coarse-grained soils between 5% and 15% fines.

BACKFILL, WELL, AND SAMPLE SYMBOLS					
Bentonite Chips		Grout		2" OD Split Barrel Sampler	
Concrete		Observation Well - Solid		Shelby Tube Sample	
Sand		Observation Well – Screen		Grab Sample	
Asphalt		Vibrating Wire Piezometer		Rock Core Run	
Gravel		Measured Groundwater Level			

DELVE underground

KEY TO SUBSURFACE LOGS

Project: City of Rainier - Water Master Plan Phase 2
Project Location: Rainier, OR
Project Number: 6401.0

Log of Boring B-1

Date(s) Drilled: **09/29/2022 - 09/29/2022**	Geotechnical Consultant: **Delve Underground**	Logged By: **A. Judy**	Checked By: **J. Quinn**
Drilling Method/ Rig Type: **Mud Rotary/Mobile B-58**	Drilling Contractor: **PLI Systems, Inc.**	Total Depth of Borehole: **71.5 ft**	
Hole Diameter: **6.00 in**	Hammer Weight/Drop (lb/in.)/Type: **140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum: **21.0 ft**	
Location: **Rainier City Marina**	Coordinates: **-13684909.32E,5794795.57N**	Elevation Source: **Pump station as-built drawings**	

ELEV. (FT)	WATER LEVEL	DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	PENETRATION RESISTANCE BLOWS/FT (10 20 30 40); WATER CONTENT (MC); ATTERBERG LL/PL (20 40 60 80)	USCS GRAPHIC	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS	BACKFILL/INSTALL.
										Pavement Section: consisting of three inches of asphalt pavement over 18 inches of base aggregate.		
				78	6-9-12 (N=21)	S-1				Medium dense, wet, gray, Poorly Graded SAND (SP); fine to medium sand. **Fill**		
16		5		83	9-15-14 (N=29)	S-2			SP			
				95	9-13-14 (N=27)	S-3						
11		10		78	5-5-9 (N=14)	S-4						
	▼									Very loose to loose, wet, gray, Silty SAND (SM); fine sand, micaceous, non-plastic. **Holocene Alluvium**	Driller notes finer-grained material at 12.5 feet. S-5: 39.5% fines by ASTM D1140.	
6		15		78	1-0-1 (N=1)	S-5						
1		20		72	1-2-3 (N=5)	S-6			SM	*Becomes medium stiff at 20 feet.*		
-4		25		89	4-1-1 (N=2)	S-7				*Becomes moist at 25.9 feet; low plastic fines from 25.9 to 30 feet.*		

DELVE underground

Boring B-1

Project: City of Rainier - Water Master Plan Phase 2
Project Location: Rainier, OR
Project Number: 6401.0

Log of Boring B-1

Date(s) Drilled: **09/29/2022 - 09/29/2022**	Geotechnical Consultant: **Delve Underground**	Logged By: **A. Judy**	Checked By: **J. Quinn**
Drilling Method/ Rig Type: **Mud Rotary/Mobile B-58**	Drilling Contractor: **PLI Systems, Inc.**	Total Depth of Borehole: **71.5 ft**	
Hole Diameter: **6.00 in**	Hammer Weight/Drop (lb/in.)/Type: **140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum: **21.0 ft**	
Location: **Rainier City Marina**	Coordinates: **-13684909.32E,5794795.57N**	Elevation Source: **Pump station as-built drawings**	

ELEV. (FT)	WATER LEVEL	DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	PENETRATION RESISTANCE BLOWS/FT (10 20 30 40); WATER CONTENT (MC); ATTERBERG LL/PL (20 40 60 80)	USCS GRAPHIC	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS	BACKFILL/INSTALL.
				100	5-5-9 (N=14)	S-8			SM	Very loose to loose, wet, gray, Silty SAND (SM); fine sand, micaceous, non-plastic. **Holocene Alluvium** *Becomes medium dense, moist, and non-plastic at 31 feet.* *Becomes orange at 31 feet.*	S-8: 26.8% fines by ASTM D1140.	
-14		35		100	1-1-6 (N=7)	S-9				Medium stiff to stiff, moist, gray, FAT CLAY (CH); high plasticity, blocky structure. **Residual Soil of Goble Volcanics** *Becomes dark brown at 36 feet.*		
-19		40		89	1-2-4 (N=6)	S-10				*Tuffaceous (trace ash and lapilli); becomes gray with orange mottling at 40 feet.*	Difficult drilling beginning at 40 feet; clay begins 'squeezing' borehole.	
-24		45		89	3-4-6 (N=10)	S-11			CH	*Becomes stiff at 45 feet; mottling becomes olive green.*		
-29		50		95	3-5-8 (N=13)	S-12				*Mottles become brown at 50 feet.* *Tuffaceous (trace ash and lapilli) at 51 feet.*		
-34		55		100	3-5-5 (N=10)	S-13				*No mottling at 55 feet.*	Clay continues constricting borehole; drillers re-drill 0.0 to 60 feet with 6-inch bit. At 60 feet bgs, drillers switch to 4-7/8-inch bit.	

Project: City of Rainier - Water Master Plan Phase 2
Project Location: Rainier, OR
Project Number: 6401.0

Log of Boring B-1

Date(s) Drilled: **09/29/2022 - 09/29/2022**	Geotechnical Consultant: **Delve Underground**	Logged By: **A. Judy**	Checked By: **J. Quinn**
Drilling Method/ Rig Type: **Mud Rotary/Mobile B-58**	Drilling Contractor: **PLI Systems, Inc.**	Total Depth of Borehole: **71.5 ft**	
Hole Diameter: **6.00 in**	Hammer Weight/Drop (lb/in.)/Type: **140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum: **21.0 ft**	
Location: **Rainier City Marina**	Coordinates: **-13684909.32E,5794795.57N**	Elevation Source: **Pump station as-built drawings**	

ELEV. (FT) / WATER LEVEL / DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	PENETRATION RESISTANCE BLOWS/FT (10 20 30 40); WATER CONTENT (MC); ATTERBERG LL/PL (20 40 60 80)	USCS GRAPHIC	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS	BACKFILL/INSTALL.
		100	4-6-8 (N=14)	S-14	■			Medium stiff to stiff, moist, gray, FAT CLAY (CH); high plasticity, blocky structure. **Residual Soil of Goble Volcanics** *Blue gray mottles at 60 feet.*		
-44 65		100	5-8-10 (N=18)	S-15	■		CH	*Becomes brown with multicolored indurated nodules, trace charcoal at 65.67 feet.*		
-49 70		83	6-10-13 (N=23)	S-16	■			*70.0 - 70.5 feet: lens of moist to wet, light gray, volcanic ash. Becomes brown, gray, and red mottled at 70.5 feet.*	Borehole completed at 71.5 feet below ground surface (bgs).	
-54 75										
-59 80										
-64 85										

Project: City of Rainier - Water Master Plan Phase 2
Project Location: Rainier, OR
Project Number: 6401.0

Log of Boring B-2

Date(s) Drilled: **09/29/2022 - 09/30/2022**	Geotechnical Consultant: **Delve Underground**	Logged By: **A. Judy**	Checked By: **J. Quinn**
Drilling Method/ Rig Type: **8.25" Hollow Stem Auger/Mobile B-58**	Drilling Contractor: **PLI Systems, Inc.**	Total Depth of Borehole: **31.5 ft**	
Hole Diameter: **8.25 in**	Hammer Weight/Drop (lb/in.)/Type: **140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum: **265.0 ft**	
Location: **750 Rainier Blvd - Upper Plant**	Coordinates: **-13684984.90E,5793736.25N**	Elevation Source: **Site Grading Plan from Construction Drawing Plans**	

ELEV. (FT)	WATER LEVEL	DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	PENETRATION RESISTANCE BLOWS/FT (10 20 30 40) / WATER CONTENT (MC) / ATTERBERG LL/PL (20 40 60 80)	USCS GRAPHIC	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS	BACKFILL/INSTALL.
										Pavement Section: consisting of two inches of asphalt pavement over 12 inches of base aggregate.		
				33	3-3-8 (N=11)	S-1	■ ○			Stiff to medium stiff, moist, orange-brown mottled, LEAN CLAY (CL); low to medium plasticity, trace fine gravel, trace fine sand, trace charcoal. **Colluvium**		
260		5		83	2-2-3 (N=5)	S-2	■ ○		CL			
255		10		100	3-3-5 (N=8)	S-3	■ ○			Medium stiff to very stiff, moist, orange with tan mottles, Sandy SILT (ML); low plasticity, fine sand, trace charcoal, micaceous. **Alluvial Terrace Deposits**		
250		15		100	2-3-3 (N=6)	S-4	■ ○			*No charcoal below 15 feet; variably indurated from 15 to 20 feet.*	S-4: 54.7% fines by ASTM D1140.	
245		20		100	1-3-3 (N=6)	S-5	■ ⊣		ML		S-5: 64.3% fines by ASTM D1140.	
240		25		100	3-3-3 (N=6)	S-6	■ ○			*Becomes tan at 25 feet.*		

DELVE underground

Boring B-2

Sheet 1

Project: City of Rainier - Water Master Plan Phase 2
Project Location: Rainier, OR
Project Number: 6401.0

Log of Boring B-2

Date(s) Drilled: **09/29/2022 - 09/30/2022**	Geotechnical Consultant: **Delve Underground**	Logged By: **A. Judy**	Checked By: **J. Quinn**
Drilling Method/ Rig Type: **8.25" Hollow Stem Auger/Mobile B-58**	Drilling Contractor: **PLI Systems, Inc.**	Total Depth of Borehole: **31.5 ft**	
Hole Diameter: **8.25 in**	Hammer Weight/Drop (lb/in.)/Type: **140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum: **265.0 ft**	
Location: **750 Rainier Blvd - Upper Plant**	Coordinates: **-13684984.90E,5793736.25N**	Elevation Source: **Site Grading Plan from Construction Drawing Plans**	

ELEV. (FT) / WATER LEVEL / DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	PENETRATION RESISTANCE BLOWS/FT (10 20 30 40) / WATER CONTENT (MC) / ATTERBERG LL/PL (20 40 60 80)	USCS GRAPHIC	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS	BACKFILL/INSTALL.
		100	8-10-9 (N=19)	S-7			ML	Medium stiff to very stiff, moist, orange with tan mottles, Sandy SILT (ML); low plasticity, fine sand, trace charcoal, micaceous. **Alluvial Terrace Deposits** *Laminated structure apparent at 30 feet.*	Borehole completed at 31.5 feet below ground surface (bgs).	

230 35
225 40
220 45
215 50
210 55

DELVE underground

Boring B-2

Project: City of Rainier - Water Master Plan Phase 2
Project Location: Rainier, OR
Project Number: 6401.0

Log of Boring B-3

Date(s) Drilled: **09/30/2022 - 09/30/2022**	Geotechnical Consultant: **Delve Underground**	Logged By: **A. Judy**	Checked By: **J. Quinn**
Drilling Method/ Rig Type: **8.25" Hollow Stem Auger/Mobile B-58**	Drilling Contractor: **PLI Systems, Inc.**	Total Depth of Borehole: **46.5 ft**	
Hole Diameter: **8.25 in**	Hammer Weight/Drop (lb/in.)/Type: **140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum: **236.0 ft**	
Location: **750 Rainier Blvd - Lower Plant**	Coordinates: **-13685035.66E,5793812.01N**	Elevation Source: **Site Grading Plan from Construction Drawing Plans**	

ELEV. (FT)	WATER LEVEL	DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	PENETRATION RESISTANCE BLOWS/FT (10 20 30 40); WATER CONTENT (MC); ATTERBERG LL/PL (20 40 60 80)	USCS GRAPHIC	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS	BACKFILL/INSTALL.
										Pavement Section: consisting of five inches of asphalt pavement over 12 inches of base aggregate.		
				100		G-1				Moist, brown-orange mottled, FAT CLAY (CH); high plasticity, slightly micaceous. **Colluvium** *Becomes light gray with black and orange mottling; no mica below 4 feet.*	2.5 - 4.0 feet bgs: material description based on augur cuttings.	
231		5		100	2-5-4 (N=9)	S-1						
226		10		100	3-6-7 (N=13)	S-2			CH			
221		15		100	3-4-5 (N=9)	S-3						
216		20		100	5-7-10 (N=17)	S-4				Medium dense, moist, gray with orange mottles, Silty SAND (SM); non-plastic, fine sand, micaceous. **Alluvial Terrace Deposits**	S-4: 38.5% fines by ASTM D1140.	
211		25		100	5-6-11 (N=17)	S-5			SM		Saturated soil cuttings observed between 28 and 40 feet indicate potential groundwater seepage at this depth interval.	

DELVE underground

Boring B-3

Sheet 1

Project: City of Rainier - Water Master Plan Phase 2
Project Location: Rainier, OR
Project Number: 6401.0

Log of Boring B-3

Date(s) Drilled: **09/30/2022 - 09/30/2022**	Geotechnical Consultant: **Delve Underground**	Logged By: **A. Judy**	Checked By: **J. Quinn**
Drilling Method/ Rig Type: **8.25" Hollow Stem Auger/Mobile B-58**	Drilling Contractor: **PLI Systems, Inc.**	Total Depth of Borehole: **46.5 ft**	
Hole Diameter: **8.25 in**	Hammer Weight/Drop (lb/in.)/Type: **140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum: **236.0 ft**	
Location: **750 Rainier Blvd - Lower Plant**	Coordinates: **-13685035.66E,5793812.01N**	Elevation Source: **Site Grading Plan from Construction Drawing Plans**	

ELEV. (FT)	WATER LEVEL	DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	PENETRATION RESISTANCE BLOWS/FT (10 20 30 40); WATER CONTENT (MC); ATTERBERG LL/PL (20 40 60 80)	USCS GRAPHIC	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS	BACKFILL/INSTALL.
				100	5-10-12 (N=22)	S-6	○ ■			Medium dense, moist, gray with orange mottles, Silty SAND (SM); non-plastic, fine sand, micaceous. **Alluvial Terrace Deposits**		
201		35		100	7-12-16 (N=28)	S-7	○ ■		SM		S-7: 18.4% fines by ASTM D1140.	
196		40		100	9-7-8 (N=15)	S-8	■		CH	Stiff to hard, moist, dark gray, FAT CLAY (CH); medium plasticity. **Residual Soil of Goble Volcanics**		
191		45		100	9-16-26 (N=42)	S-9	■			*Becomes hard at 45 feet.*		
186		50									Borehole completed at 46.5 feet below ground surface (bgs).	
181		55										

Appendix B

Laboratory Test Results

Breccia Geotechnical Testing, LLC.	Natural Moisture Content (ASTM D2216)

Client:	McMillen Jacobs Associates	By:	FS
Project Name:	City of Rainier - Water Master Plan Phase 2	Date:	10/15/2022
Project Number:	6401.0		

Exploration ID	B-1	B-1	B-1			
Samples ID	S-1	S-3	S-4			
Samples Depth (ft.)	2.5-4	7.5-9	10-11.5			
Moisture Content (%)	22.4	25.6	25.3			

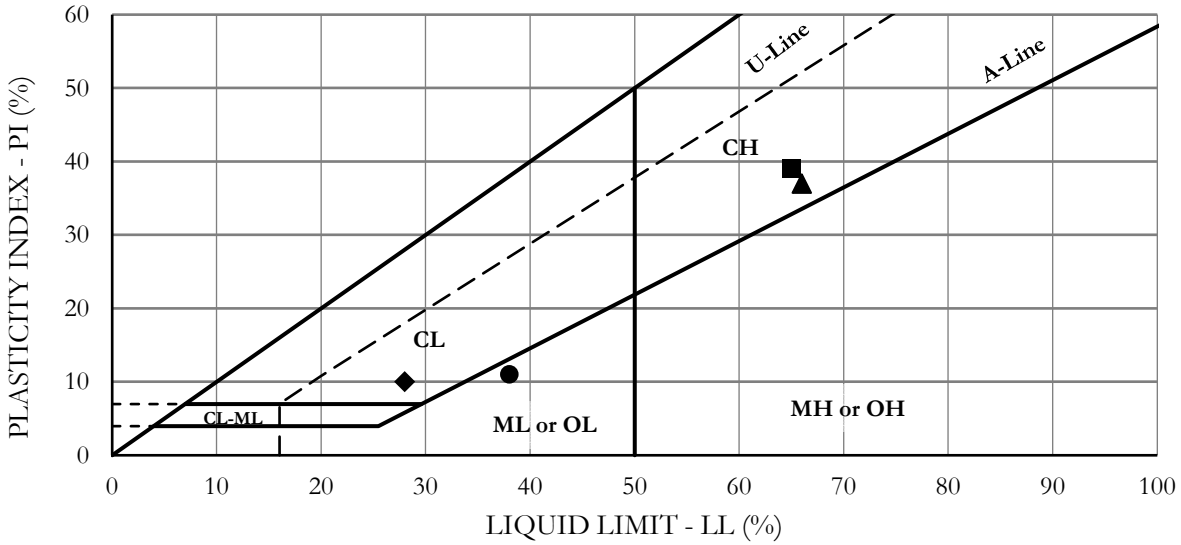
Exploration ID	B-2	B-2	B-2	B-2	B-2	
Samples ID	S-1	S-2	S-3	S-6	S-7	
Samples Depth (ft.)	2.5-4	5-6.5	10-11.5	25-25.9	30-31.5	
Moisture Content (%)	31.6	36.0	37.3	33.4	33.4	

Exploration ID	B-3	B-3	B-3	B-3	B-3	
Samples ID	S-1	S-2	S-3	S-5	S-6	
Samples Depth (ft.)	5-6.5	10-11.5	15-16.5	25-26.5	30-31.5	
Moisture Content (%)	52.1	35.6	43.2	31.9	30.7	

Breccia Geotechnical Testing, LLC.		Percent Fines (ASTM D1140)	
Client:	McMillen Jacobs Associates	By:	FS
Project Name:	City of Rainier - Water Master Plan Phase 2	Date:	10/16/2022
Project Number:	6401.0		

Exploration ID	B-1	B-1	B-2	B-2	B-3	B-3
Samples ID	S-5	S-8	S-4	S-5	S-4	S-7
Samples Depth (ft.)	15-16.5	30-31.5	15-16.5	20-21.5	20-21.5	35-36.5
Moisture Content (%)	43.3	33.5	34.2	34.3	25.0	33.5
Percent Fines (%)	39.5	26.8	54.7	64.3	38.5	18.4

ATTERBERG LIMITS TEST RESULTS (ASTM D4318)



	Boring	Sample ID	Depth (feet)	Moisture Content (%)	Atterberg Limits			%Pass #200	USCS
					LL	PL	PI		
◆	B-1	S-7b	25.9-26.5	33.4	28	18	10	--	CL
▲	B-1	S-9	35-36.5	40.7	66	29	37	--	CH
●	B-2	S-5	20-21.5	34.3	38	27	11	64.3	ML
■	B-3	GS-1	3-4	44.0	65	26	39	--	CH
◇									
△									
○									

Remarks

Project: City of Rainier - Water Master Plan Phase 2
Project No.: 6401.0
Location: Rainier, OR

Breccia Geotechnical Testing, LLC.
Brecciageolab@gmail.com
Tel: 971-246-1324

Appendix C

Fox Creek Reservoir Dam Photos and Planning Level Cost Estimate for Improvements

Photo 1: Downstream face of dam.

Photo 2: Closeup view of downstream face of dam.

Photo 3: Right abutment condition of the dam.

Photo 4: Left abutment condition of the dam.

Photo 5: Wood cribs and grade-beams below the dam facing.

Photo 6: Wood cribs and grade-beams in the left abutment area.

Photo 7: Upstream face of the dam (provided by the City).

Photo 8: Upstream face of dam and outlet gate (provided by the City).

Item	Estimated Quantity		Unit Price	Total
Mobilization	1	LS	5%	$ 162,000
Access Improvement	1	LS	$ 700,000	$ 700,000
Existing Foundation Anchorage	1	LS	$ 300,000	$ 300,000
Subgrade Overexcavation	1	LS	$ 200,000	$ 200,000
Outlet Pipe and Gate Replacement	1	LS	$ 350,000	$ 350,000
Concrete Filling of Existing Dam	3800	CY	$ 300	$ 1,140,000
Spillway Improvement	1	LS	$ 150,000	$ 150,000
Dredging and Clearing	1	LS	$ 100,000	$ 100,000
Impermeable Liner (upstream face and abutments)	1	LS	$ 300,000	$ 300,000
Construction Subtotals[1, 2]				**$ 3,402,000**
Design	20%			$ 681,000
Engineering Services during Construction	5%			$ 170,100
CM and Inspection	10%			$ 340,200
Totals (before Contingency)				**$ 4,593,300**
Contingency	40%			$ 1,360,800
Estimated Cost				**$ 5,954,100**

Notes:
1. All construction costs in 2021 dollars; Class IV estimates.
2. Fish passage and habitat improvment are assumed to not be required for this project.

Table 3: Planning Level Cost Estimate

Appendix B

Seismic Evaluation of Existing Structures Technical Memorandum – May 2, 2023

By Ace Engineering LLC

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TECHNICAL MEMORANDUM

DATE: **May 2, 2023**

TO: **WEST YOST**

ATTENTION: **SANDRINE GANRY**

PROJECT: **2022-08, CITY OF RAINIER, OREGON, WATER MASTER PLAN**

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

1.0 Introduction

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

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

1. Review existing documentation of each structure that was made available by the City.
2. Review Seismic Hazards Evaluation prepared by McMillen Jacobs Associates, March 5, 2021.
3. Site observation of each significant structure in the water treatment and distribution system on September 12, 2022.
4. Abbreviated description of the structural system of each significant structure in the water treatment and distribution system.
5. Complete ASCE/SEI 41-17 Tier 1 Checklists, Quick Checks, and Evaluations.
6. Abbreviated summary of findings and identification of shortcomings of each significant structure in the water treatment and distribution system.

2.0 Documentation Review

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

1. Columbia River Raw Water Intake (1998)
2. Water Treatment Plant – Filter Building (1998)
3. Water Treatment Plant – Tank (1998)
4. Water Treatment Plant – Backwash Lagoon (1998)
5. Old Rainier Hwy Tank (unknown)
6. Townsend Tank (2009)
7. Neer City Road Tank (2000)

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

3.0 Site Observation

Each significant structure of the water treatment and distribution system was observed on September 12, 2022. Sue Lawrence and John Dewey of the City of Rainier were present during most of the site observations. The existing structures were observed for compliance with the original design drawings and details. Deviations from the original design documents were noted. Signs for structural deficiencies or distress were a primary focus and any signs were noted.

4.0 Structure Summaries

4.1 Columbia River Raw Water Intake

The Raw Water Intake structure is located on the south bank of the Columbia River. The intake structure consists of a slab on grade cast on a precast concrete manhole with 8" CMU block walls supporting a wood framed roof. The structure was designed in 1998 and is in good condition. The concrete slab foundation is exposed at the base of the building.

4.2 Water Treatment Plant – Filter Building

The Filter Building consists of a slab on grade with 8" CMU block walls supporting a wood framed roof. The north portion of the building contains a wood framed mezzanine above the laboratory, restroom & electrical room. There is a loading dock on the west side of the building and a generator room on the north side. The structure was designed in 1998 and is in good condition.

4.3 Water Treatment Plant – Tank

The Tank at the Water Treatment Plant is a welded steel tank on a concrete foundation on grade that was designed in 1998. The steel tank rests on the concrete foundation and is held in place by friction between the steel base and the concrete foundation. No anchor bolts are present. There are a few areas of moss growth on the roof & walls. There are a few minor paint chips, flakes or scratches on the roof & walls. The structure is in good condition.

4.4 Water Treatment Plant – Backwash Lagoon

The Backwash Lagoon is just west of the Filter Building. The Lagoon is a concrete structure that was designed in 1998 and is in good condition.

4.5 Old Rainier Hwy Tank

The Tank on Old Rainier Hwy is a welded steel tank on a concrete foundation on grade that was designed in 2000. The steel tank rests on the concrete foundation and is held in place by friction between the steel base and the concrete foundation. No anchor bolts are present. There are a few areas of moss growth on the roof & walls. There are a few minor paint chips, flakes or scratches on the roof & walls. The structure is in good condition.

4.6 Townsend Tank

The Tank on Townsend Road is a welded steel tank on a concrete foundation on grade that was designed in 2009. The steel tank rests on the concrete foundation and is held in place by stainless steel anchor bolts epoxied into the concrete foundation to prevent overturning. The anchor bolts appear to resist overturning only, so sliding forces must be resisted by friction between the tank base and the concrete foundation. There are many areas of moss growth on the roof & walls. There are many areas where the paint has blistered, chipped or flaked on the

roof & walls where rust is occurring. There is no loss to the thickness of the steel at this point, but if this corrosion is not addressed it may become a structural concern. Overall, the structure is in good condition.

4.7 Neer City Road Tank

The Tank on Neer City Road is a welded steel tank on a concrete foundation on grade that was designed in 2000. The steel tank rests on the concrete foundation and is held in place by friction between the steel base and the concrete foundation. No anchor bolts are present. There are a few areas of moss growth on the roof & walls. There are a few minor paint chips, flakes or scratches on the roof & walls. The structure is in good condition.

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

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

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

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

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

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

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

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

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

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

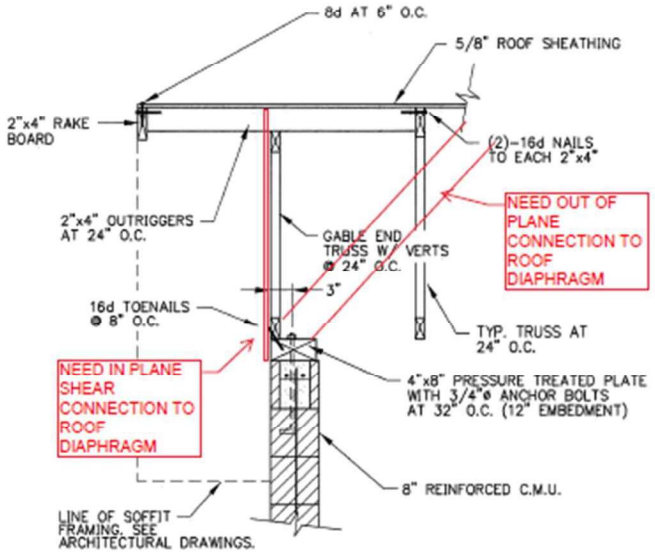
6.0 Seismic Rehabilitation Recommendations

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

6.1 Columbia River Raw Water Intake

The Raw Water Intake structure is considered a Reinforced Masonry Bearing Walls with Flexible Diaphragm (RM1) structure. The noncompliant items discovered in the checklists, document review and site observation include:

- WALL ANCHORAGE: The roof diaphragm is not connected to the CMU block walls to adequately resist out of plane forces on the gable sides of the structure.
- TRANSFER TO SHEAR WALLS: The roof diaphragm does not appear to be directly connected to the CMU block walls on the gable sides of the structure.
- DEEP FOUNDATIONS: The structure is placed on top of a precast concrete manhole. The manhole is placed in sections. The sections have been bolted together, but it is unclear if the manhole is capable of transferring lateral forces from the structure to the soil.
- SLOPING SITES: The site slopes dramatically down to the river. The Geotechnical Engineer should assess if the soil is capable of providing lateral resistance for the structure and manhole.

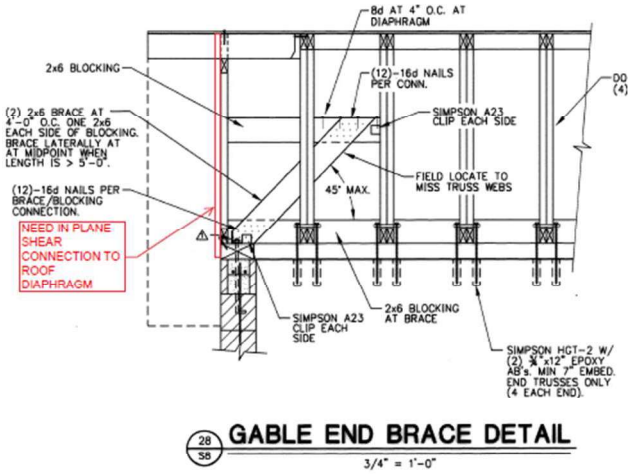


- The downspout on the back side of the structure should be routed downhill away from the foundation.
- The bottom of the concrete slab foundation is exposed on nearly all sides. Minimum frost depth and lateral resistance is typically 12 inches.

6.2 Water Treatment Plant – Filter Building

The Filter Building is considered a Reinforced Masonry Bearing Walls with Flexible Diaphragm (RM1) structure. The noncompliant items discovered in the checklists, document review and site observation include:

- TRANSFER TO SHEAR WALLS: The roof diaphragm does not appear to be directly connected to the CMU block walls on the gable sides of the structure.
- SUSPENDED CONTENTS: Piping and vacuum lines are suspended from the structure and are free to swing or move, but may damage themselves or adjoining components.
- TALL NARROW EQUIPMENT: There are several pieces of equipment, like the compressor, more than 6 feet tall that should be anchored to the floor or adjacent walls.



- CONDUIT COUPLINGS: Conduit greater than 2.5 inches should have flexible couplings.
- The roof gutters appear to leak at the joints causing the fascia to rot. If left unmaintained the structure will be damaged and need to be replaced.
- The downspouts are terminated very near the base of the walls. The downspouts should be routed away from the structure to prevent erosion and fungus build up on the structure.
- Efflorescence was observed on the west side of the structure in the concrete wall near the stairs to the loading dock. This typically occurs when moisture is transmitting through the concrete wall. It may be possible that water is leaking inside the building and migrating to the west. If moisture continues to transmit through the wall structural damage will result. It is recommended that the trench & piping inside the Filter Building be checked for leaks and water inside the building routed to reliable outlets.
- The rust and corrosion around the base of the steel pipe supports should be treated, repaired and properly coated to prevent further corrosion.
- The sodium hypochlorite tank should have clips bolted to the concrete slab to prevent lateral movement and overturning during a seismic event.
- The caustic soda tank should have clips bolted to the concrete slab to prevent lateral movement and overturning during a seismic event.

6.3 Water Treatment Plant – Tank

The Tank at the Water Treatment Plant is considered a Steel Plate Shear Wall (S6) structure. The noncompliant items discovered in the checklists, document review and site observation include:

- It is recommended that the steel tank be bolted to the concrete foundation to prevent sliding and overturning during a seismic event.
- Clean and remove any moss, mold, rust and corrosion. Then touch up paint any areas to prevent loss of steel thickness.

6.4 Water Treatment Plant – Backwash Lagoon

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

6.5 Old Rainier Hwy Tank

The Tank on Old Rainier Hwy is considered a Steel Plate Shear Wall (S6) structure. The noncompliant items discovered in the checklists, document review and site observation include:

- It is recommended that the steel tank be bolted to the concrete foundation to prevent sliding and overturning during a seismic event.
- Clean and remove any moss, mold, rust and corrosion. Then touch up paint any areas to prevent loss of steel thickness.

6.6 Townsend Tank

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

- It is recommended that the steel tank be bolted to the concrete foundation to prevent sliding during a seismic event.
- Remove the rust and corrosion then clean and paint the tank to prevent corrosion and loss of steel thickness.

6.7 Neer City Road Tank

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

- It is recommended that the steel tank be bolted to the concrete foundation to prevent sliding and overturning during a seismic event.
- Clean and remove any moss, mold, rust and corrosion. Then touch up paint any areas to prevent loss of steel thickness.

6.8 General nonstructural items.

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

- FIRE SUPPRESSION PIPING: Make sure piping is anchored and braced in accordance with current NFPA standards. Consider anchoring and bracing all piping in all facilities.
- HAZARDOUS MATERIAL STORAGE: Some chemicals used in the treatment process or used during regular cleaning and maintenance processes may be considered hazardous when spilled. Items storing these chemicals should be restrained to prevent displacement, tipping or falling.
- HAZARDOUS MATERIAL DISTRIBUTION: Natural gas piping should anchored or braced adequately to prevent damage that might allow the hazardous material to release.
- SHUTOFF VALVES: Piping containing hazardous material, including natural gas, should have shutoff valves or other devices to prevent spills or leaks.
- FLEXIBLE COUPLINGS: Hazardous material, ductwork and piping, including natural gas piping, should have flexible couplings.

- LIGHT FIXTURES LENSE COVERS: Make sure lens covers on light fixtures are attached with safety devices and add safety devices if necessary.
- INDUSTRIAL STORAGE RACKS: Industrial storage racks or similar items that are more than 12 feet high should be anchored to the floor.
- TALL NARROW CABINETS: Cabinets, lockers, bookshelves, etc. more than 6 feet high and with height-to-depth ratios exceeding 3:1 should anchored to the floor or wall.
- FALL-PRONE CONTENTS: Equipment, stored items weighing more than 20 pounds and more than 4 feet above the floor should be braced or restrained.
- FALL-PRONE EQUIPMENT: Equipment weighing more than 20 pounds and more than 4 feet above the floor should be braced or restrained.
- IN-LINE EQUIPMENT: Equipment installed in line with a duct or piping system, with an operating weight more than 75 pounds should be laterally braced independent of the duct or piping system.
- TALL NARROW EQUIPMENT: Equipment, tanks, etc. more than 6 feet high and with height-to-depth ratios exceeding 3:1 should anchored to the floor or wall.
- SUSPENDED EQUIPMENT: Equipment suspended without lateral bracing should be free to swing or move with the structure without damaging itself or adjoining components.
- HEAVY EQUIPMENT: Floor supported or platform supported equipment weighing more than 400 pounds should be anchored to the structure.
- CONDUIT COUPLINGS: Conduit greater than 2.5 inches should have flexible couplings.
- FLEXIBLE COUPLINGS: Fluid and gas piping should have flexible couplings.
- FLUID AND GAS PIPING: Fluid and gas piping should be anchored and braced to the structure to limit spills or leaks.

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

7.0 Exceptions

7.1 Townsend Road Wood Tank

The 1975 wood tank at the Townsend Road site is not in use for the water distribution system and was not included in this evaluation.

7.2 Fox Creek Dam

The wood dam that creates the reservoir on Fox Creek is not used for the water distribution system and was not included in this evaluation.

8.0 Conclusions

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

9.0 Limitations

This Structural Technical Memorandum has been prepared for the City of Rainier Water Master Plan. The conclusions and recommendations in this memorandum were derived from the professional review of documentation that was provided by the City of Rainier, West Yost,

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published literature and limited site observations. ACE Engineering is not responsible for errors and omissions that might exist in documents and construction performed by others.

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

ACE ENGINEERING LLC

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

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

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Table 17-3. Immediate Occupancy Basic Configuration Checklist

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

continues

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

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

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

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

COLUMBIA RIVER RAW WATER INTAKE STRUCTURE

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

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

continues

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

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

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

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WATER TREATMENT PLANT - FILTER BUILDING

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

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

continues

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

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

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WTP TANK

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

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

continues

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

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

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OLD COLUMBIA RIVER HWY TANK

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

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

continues

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

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

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TOWNSEND TANK

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

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

continues

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

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

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NEER CITY TANK

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

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

continues

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

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

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

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Table 17-38. Nonstructural Checklist

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

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

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

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

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

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

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

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

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

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

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

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

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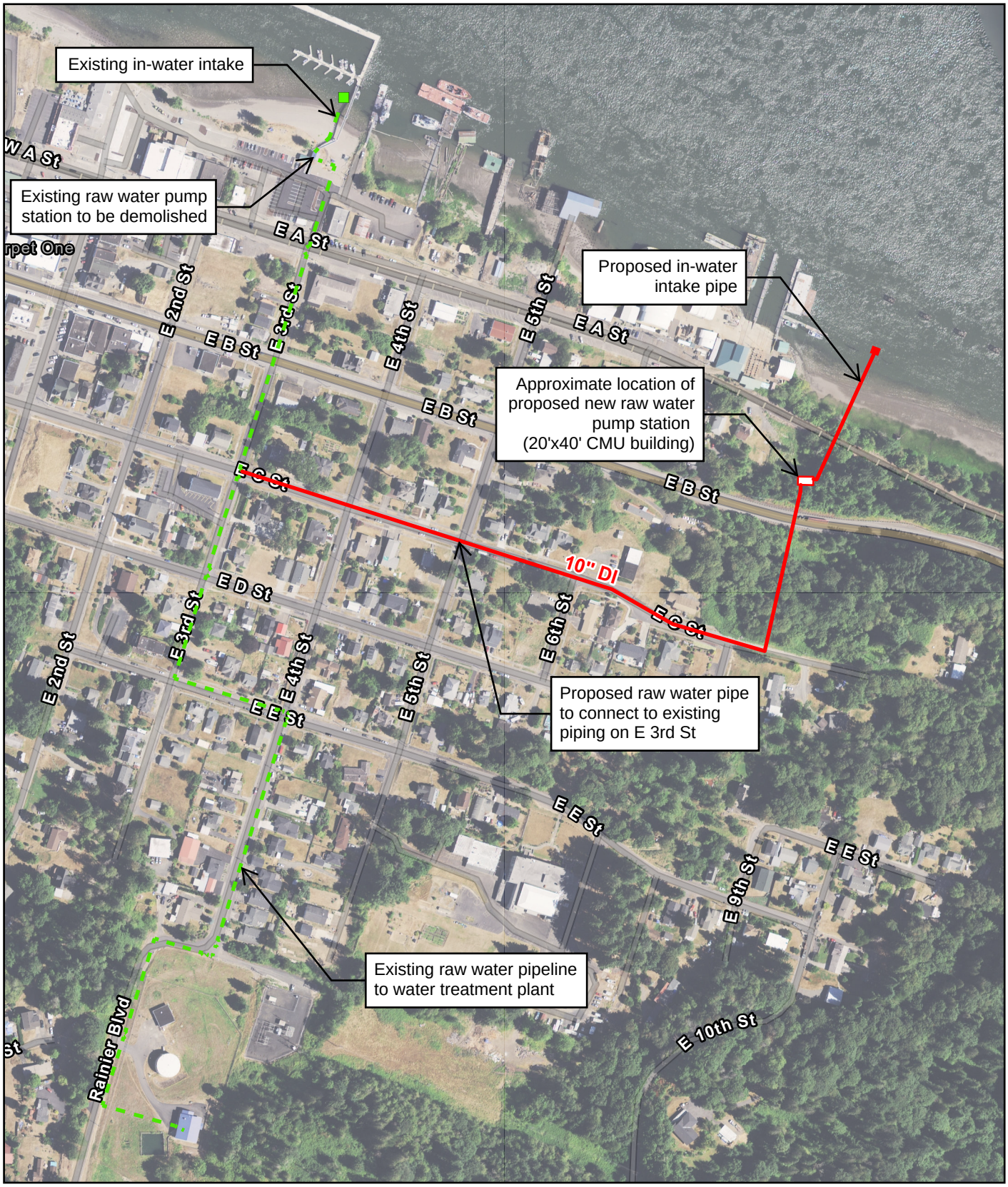
Appendix C

Columbia River Raw Water Intake Facility – Preliminary Cost Estimate and Proposed New Intake Facility Layout

Raw Water Pump Station Cost Estimate	
Line Item Cost	Estimated Cost, dollars
Mobilization, Demobilization	170,000
Civil and Sitework	310,000
Concrete	450,000
CMU Building (20'x40')	370,000
Mechanical Equipment and Piping	530,000
Electrical, Instrumentation and Controls	370,000
In-water intake pipe/screen	650,000
Pipe to connect to ex. raw water piping	450,000
Demolition of existing pump station	150,000
Subtotal	**$2,850,000**
Plus: General Conditions - 12 percent	340,000
Plus: Contractor Overhead and Profit - 15 percent	430,000
Plus: Escalation to Mid-Point of Construction - 20 percent	570,000
Subtotal	**$4,190,000**
Engineering, Legal & Administrative - 25 percent	1,050,000
Land Acquisition	150,000
Permitting - 10 percent	420,000
Contingency - 40 percent	1,680,000
Total	**$7,490,000**



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Scale in Feet

Figure C-1

Preliminary Layout for New Columbia River Raw Water Intake Facility

City of Rainier
Water Master Plan Update
Appendix C