Joint Water Commission

WATER SYSTEM MASTER PLAN

FINAL | June 2023

Joint Water Commission

WATER SYSTEM MASTER PLAN

FINAL | June 2023

Digitally signed by Alena M. Thurman
Contact Info: Carollo Engineers, Inc.
Date: 2023.06.26 13:27:51-07'00'

Alena M Thurman

REGISTERED PROFESSIONAL
ENGINEER
94763
OREGON
MAY 14, 2019
ALENA M. THURMAN
EXPIRES: 06/30/24

Contents

Executive Summary

Chapter 1 - Introduction

Chapter 2 - Existing System

Chapter 3 - Planning Considerations

Chapter 4 - Water Quality and Regulatory Framework

Chapter 5 - Water Demand Forecast and Supply Strategy

Chapter 6 - Intake Evaluation

Chapter 7 - WTP Evaluation

Chapter 8 - Transmission System Evaluation

Chapter 9 - Capital Improvement Program

Appendices

Tables

Figures

Abbreviations

°C	degrees Celsius
$k	thousand US dollars
$M	million US dollars
3D	three-dimensional
A	ampere(s)
AACE	Association for the Advancement of Cost Engineering
AC	asbestos cement
ac-ft	acre-feet
ADD	average day demand
AFY	acre-feet per year
AI	Aggressiveness Index
AIC	Amps interrupting capacity
ALA	American Lifelines Alliance
alum	aluminum sulfate
ASCE	American Society of Civil Engineers
ASR	aquifer storage and recovery
ATS	automatic transfer switch
Ave	Avenue
AWWA	American Water Works Association
Beaverton	City of Beaverton
BFE	base flood elevation
Bldg. 8	(Original) Generator Building
BRIC	Building Resilient Infrastructure and Communities
BRJOC	Barney Reservoir Joint Ownership Commission
BSE-1N	Basic Safety Earthquake-1
BSE-2N	Basic Safety Earthquake-2
BW	backwash
C	Celsius
$CaCO_3$	calcium carbonate
Carollo	Carollo Engineers, Inc.
caustic soda	sodium hydroxide
CB	circuit breaker
CCCIS	cell-to-cell close internal survey
CCCS	Cascade Corrosion Consulting Services
CCE	construction cost estimate
CCP	concrete cylinder pipe
CCPP	Calcium Carbonate Precipitation Potential
CCTV	closed-circuit television

CDC	Community Development Code
CEC	contaminants of emerging concern
cf	cubic feet
CFE	combined filter effluent
cfm	cubic feet per minute
cfm/sf	cubic feet per minute per square foot
cfs	cubic feet per second
CI	cast iron
CIMP	capital improvement maintenance projects
CIP	capital improvement program
CIS	close interval survey
Cl_2	chlorine
CLOMR	conditional letter of map revision
CML&C SP	cement mortar lined and coated steel pipeline
Cornelius	City of Cornelius
CP	cathodic protection
CSMR	chloride-to-sulfate mass ratio
CSZ	Cascadia Subduction Zone
CT	contact time
CWS	Clean Water Services
DBP	disinfection byproduct
DBPR	Disinfectants and Disinfection Byproduct Rule
DC	direct current
DEQ	Oregon Department of Environmental Quality
DHA	drainage hazard area
DI	ductile iron
DIC	dissolved inorganic carbon
DOC	dissolved organic carbon
DOE	Department of Energy
DOGAMI	Oregon Department of Geology and Mineral Industries
DSL	Oregon Department of State Lands
ea	each
EFU	exclusive farm use
Ele	electrical
EPA	U.S. Environmental Protection Agency
ERDIP	Earthquake Resilient Ductile Iron Pipes
ESA	Endangered Species Act
ESDC	Engineering Services During Construction
FEMA	Federal Emergency Management Act
FG	Forest Grove (transmission line)
FGHB	Forest Grove-Hillsboro (transmission line)

FIRM	flood insurance rate map
floc/sed	flocculation and sedimentation
FMA	Flood Mitigation Assistance
Forest Grove	City of Forest Grove
ft	foot/feet
ft/s	feet per second
FW	finished water
FWP	finished water pipeline
FWPS	finished water pump station
FWPS1	finished water pump station number 1
FWPS2	finished water pump station number 2
FWPS3	finished water pump station number 3
FY	fiscal year
g	multiples of acceleration due to gravity
gph	gallons per hour
gpm	gallons per minute
gpm/sf	gallons per minute per square foot
HAA5	a group of five haloacetic acids (monochloroacetic acid, dichloroacetic acid, trichloroacetic acid, monobromoacetic acid, and dibromoacetic acid)
HAA6Br	a group of six brominated haloacetic acids (bromochloroacetic acid, bromodichloroacetic acid, dibromoacetic acid, dibromochloroacetic acid, monobromoacetic acid, and tribromoacetic acid)
HAA9	a group of nine haloacetic acids (bromochloroacetic acid, bromodichloroacetic acid, chlorodibromoacetic acid, dibromoacetic acid, dichloroacetic acid, monobromoacetic acid, monochloroacetic acid, tribromoacetic acid, trichloroacetic acid)
HAL	health advisory level
HDD	horizontal directional drill
HDPE	high-density polyethylene
HGL	hydraulic grade line
Hillsboro	City of Hillsboro
HMGP	Hazard Mitigation Grant Program
hp	horsepower
hwy	highway
I&C	instrumentation and control
ICCT	interim corrosion control treatment
IEEE	Institute of Electrical and Electronics Engineers
IFA	Business Oregon Infrastructure Finance Authority
IMU	inertial measurement unit
IGA	intergovernmental agreement
IOC	inorganic contaminant

JPA	Joint Permit Application
JWC	Joint Water Commission
K_1, K_2	material constants
kW	kilowatt
kWh	kilowatt hour
L	liter
lb/day	pounds per day
lb/hr	pounds per hour
lb/sf	pounds per square foot
LCR	Lead and Copper Rule
LCRR	Lead and Copper Rule Revisions
LF	linear feet
LI	Larson's and Modified Larson's Ratio
LiDAR	Light and Radar
LL	limited license
LOMR	letter of map revision
LOS	level of service
LRAA	locational running annual average
LSI	Langelier Saturation Index
LT2ESWTR	Long Term 2 Enhanced Surface Water Treatment Rule
M9.0	magnitude 9.0
MBTA	Migratory Bird Treaty Act
MCC	motor control center
MCE	Maximum Considered Earthquake
MCE_R	Risk-Targeted Maximum Considered Earthquake
MCL	maximum contaminant level
MDD	maximum day demand
Mech	mechanical
µg/L	micrograms per liter
MG	million gallons
mgd	million gallons per day
mg/L	milligrams per liter
MIB	2-Methylisoborneol
MJA	McMillen Jacobs Associates
mL	milliliter
MLCS	mortar-lined and coated-steel pipeline
mm	millimeter
MPN	most probable number
MPZ	Mini-Power Zone
mV	millivolts
MV	medium voltage

MW	megawatt
N	nitrogen
N-S Intertie	north-south intertie
N/A	not applicable
NCOD	National Contaminant Occurrence Database
NDMA	N-Nitrosodimethylamine
NEMA	National Electrical Manufacturers Association
NFIP	National Flood Insurance Program
ng/L	nanograms per liter
NIST	National Institute of Standards and Technology
NMFS	National Marine Fisheries Service
NOAA	National Oceanic and Atmospheric Administration
North Plains	City of North Plains
NPV	net present value
NRCS	U.S. Department of Agriculture Natural Resources Conservation Services
NSF	National Sanitation Foundation
NTL	north transmission line
NTU	nephelometric turbidity unit
O&M	operations and maintenance
OAR	Oregon Administrative Rules
ODFW	Oregon Department of Fish and Wildlife
OEM	Oregon Department of Energy Management
OH&P	overhead and profit
OHA	Oregon Health Authority
OHW	ordinary high water
OPS	operations
ORP	Oregon Resilience Plan
ORS	Oregon Revised Statute
OSHG	on-site hypochlorite generation
OSSC	Oregon Structural Specialty Code
OWRD	Oregon Water Resources Department
PAC	powdered activated carbon
PEC	pulsed eddy current
PF	peaking factor
PFAS	per- and polyfluoroalkyl substances
PFOA	perfluorooctanoic acid
PFOS	perfluorooctanesulfonic acid
PGA	peak ground acceleration
PGD	permanent ground deformation
PGE	Portland General Electric
PGV	peak ground velocity

Plan	JWC Master Plan
PLW 2.0	Cornelius Pass Road Pipeline
PMR	physical map revision
PNL	panelboard
POD	points of diversion
ppb	parts per billion
ppd	pounds per day
PQL	practical quantitation limit
PRV	pressure-reducing valve
PS	pump station
PSCIS	pipe-to-soil close internal survey
PSHA	probabilistic seismic hazard analysis
psi	pounds per square inch
psig	pounds per square inch gauge
PWB	Portland Water Bureau
RFEC/TC	remote field eddy current/transformer coupling
rpm	revolutions per minute
RR	repair rate (repairs per 1,000 feet)
RW	raw water
RWP	raw water pipeline
SCADA	supervisory control and data acquisition
scfm	standard cubic feet per minute
scfm/sf	standard cubic feet per minute per square foot
SDWA	Safe Drinking Water Act
SDWRLF	Safe Drinking Water State Revolving Loan Fund
sec	second(s)
sf	square foot/feet
SFHA	special flood hazard area
SHPP	Spring Hill Pumping Plant
SHPO	State Historic Preservation Office
SLOPES	Standard Local Operating Procedures for Endangered Species
SNR	Significant Natural Resources
SOC	synthetic organic contaminant
SPDG	Salmonid Passage Design Guidelines
SPIRE	State Preparedness and Incident Response Equipment
STL	south transmission line
Struct	structural
SWBD	switchboard
SWGR	switchgear
TBD	to be determined
TDH	total dynamic head

TDS	total dissolved solids
Tigard	City of Tigard
TM	technical memorandum
TOC	total organic carbon
TON	threshold odor number
TS	test station
TTHM	total trihalomethanes (bromodichloromethane, dibromochloromethane, bromoform, and chloroform)
TV	Tualatin Valley
TVID	Tualatin Valley Irrigation District
TVWD	Tualatin Valley Water District
UCM	EPA Unregulated Contaminant Monitoring Program
UCMR	Unregulated Contaminants Monitoring Rule
UCMR3	Unregulated Contaminants Monitoring Rule, round 3
UCMR4	Unregulated Contaminants Monitoring Rule, round 4
UCMR5	Unregulated Contaminants Monitoring Rule, round 5
US	United States
USACE	U.S. Army Corps of Engineers
USBOR/USBR	United States Bureau of Reclamation
USFWS	U.S. Fish and Wildlife Service
USGS	United States Geological Survey
VAC	volts alternating current
V	volt
VFD	variable frequency drive
VOC	volatile organic contaminant
WADD	winter average daily demand
WIFIA	Water Infrastructure Finance Innovation Act
WMCP	Water Management and Conservation Plan
WQP	water quality parameters
WRWTP	Willamette River Water Treatment Plant
WWSS	Willamette Water Supply System
WWTP	wastewater treatment plant
WTP	water treatment plant
XFMR	transformer

EXECUTIVE SUMMARY

ES.1 Introduction

Established in 1976, the Joint Water Commission (JWC) is an intergovernmental water supply agency formed under an Oregon Revised Statue (ORS) 190 agreement between the Cities of Hillsboro (Hillsboro), Forest Grove (Forest Grove), Beaverton (Beaverton), and the Tualatin Valley Water District (TVWD). With ownership shared between the four member agencies, JWC acts as the primary drinking water supplier to approximately 450,000 customers in the growing communities of Washington County, Oregon.

To remain ahead of customer demand and uphold a safe and reliable system, JWC maintains a living capital improvement program (CIP) within a water master plan. JWC updates this master plan regularly to diagnose the current condition, capacity, and resilience of their water system and establish a 20-year roadmap for successful operations and capital improvements.

This 2023 *Water System Master Plan* (Plan) updates its preceding iteration, *JWC Capital Improvement Master Planning Services: 2009 Master Plan*, and prepares a robust CIP that encompasses a 20-year planning horizon from fiscal year (FY) 2023/24 through FY 2042/43. Resilience projects are spread out over a 40-year planning horizon from FY 2023/24 through FY 2062/63. Projects within the CIP were strategically identified as part of the analyses performed during development of this Plan and carried over from previous studies.

The JWC Board of Commissioners authorized the Master Plan's preparation in accordance with all JWC policies and procedures and all applicable federal, state, and county rules and regulations. The plan also complies with Chapter 333-061 of the Oregon Administrative Rules, per the Oregon Health Authority (OHA).

ES.2 Existing System

JWC's water supply is sourced from the Tualatin River and the Middle Fork of the Trask River's North Fork. To draw, treat, and distribute water from these sources, JWC employs several critical assets, including the Spring Hill Pumping Plant (SHPP) and river intake, the JWC Water Treatment Plant (WTP), and a transmission system with interconnections to JWC partners and wholesale customers. JWC's wholesale water customers include the City of North Plains (North Plains) and Westside Lutheran School. Water is also wheeled through the City of Hillsboro to serve the City of Gaston, L.A. Water Cooperative, and City of Cornelius (Cornelius).

Figure ES.2 shows a map of the JWC water system.

ES.2.1 Spring Hill Pumping Plant

Owned by the United States Bureau of Reclamation (USBOR), the 88-million-gallon-per-day (mgd) capacity SHPP serves as the JWC WTP's intake and pump station on the Tualatin River. The intake draws raw water, which passes through a series of screens before being pumped via two pipelines to the JWC WTP.

ES.2.2 JWC Water Treatment Plant

Located along the Tualatin River, the JWC WTP is a conventional media-filtration plant that employs coagulation, flocculation and sedimentation, filtration, and disinfection to produce high-quality drinking water. Originally constructed in 1976, this facility has undergone several major upgrade projects between 1995 and 2020. The most recent expansion project removed hydraulic bottlenecks that increased the plant's maximum capacity from 75 mgd to 85 mgd.

Figure ES.1 shows the JWC WTP's current site plan.

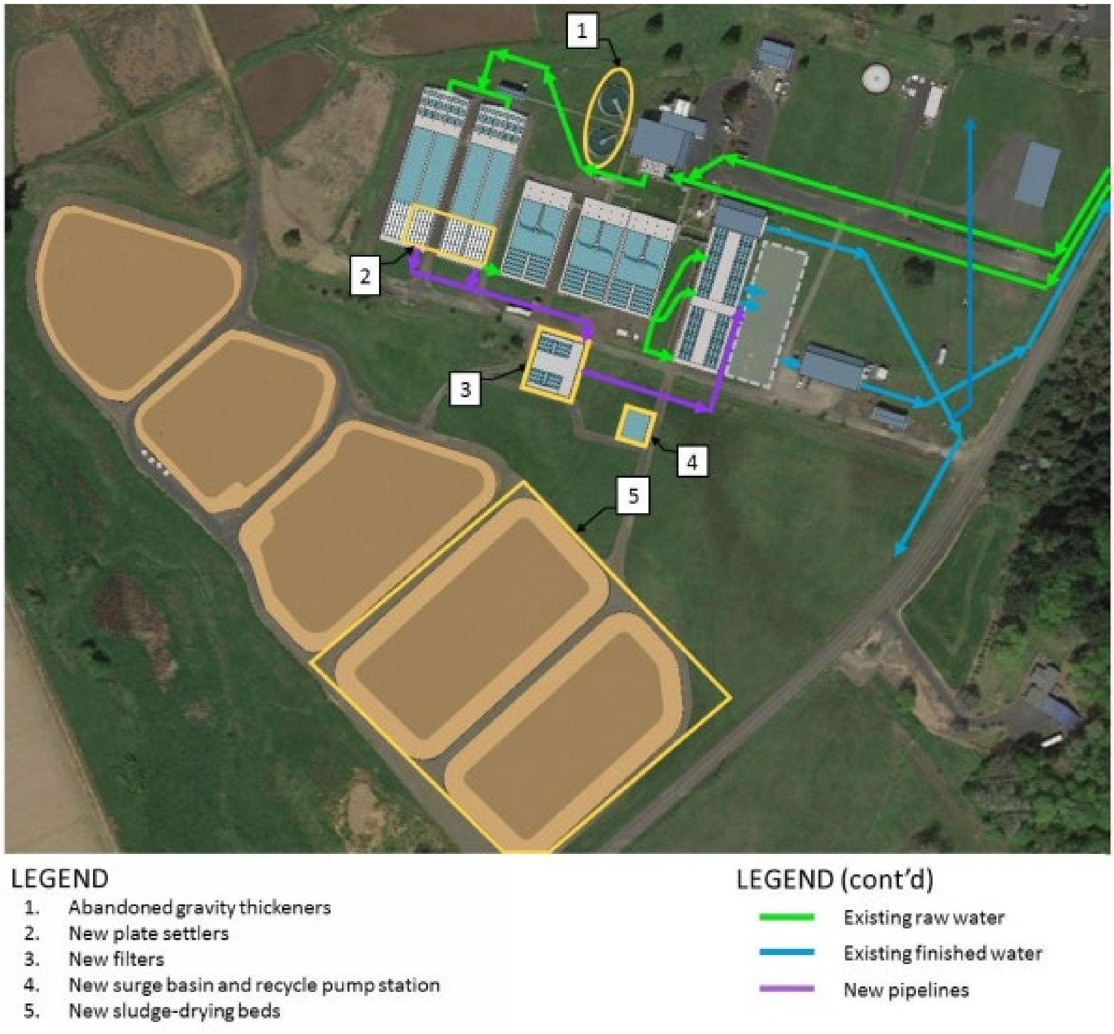


Figure ES.1 JWC WTP Site Plan

ES.2.3 Transmission System

Finished water from the JWC WTP is stored in the two Fern Hill Reservoirs, each with a capacity of 20 million gallons (MG), to maximize storage, meet the plant's operating requirements, and respond to emergencies. Water is then distributed to JWC's member agencies via the following three transmission lines:

- **North Transmission Line (NTL)**: Approximately 74,200-linear-feet (LF) long and serves Hillsboro, North Plains, Cornelius, and TVWD.
- **South Transmission Line (STL)**: Approximately 64,000-LF long and serves Hillsboro, TVWD, and Beaverton.
- **Forest Grove-Hillsboro transmission main (FGHB)**: Approximately 8,200-LF long and feeds Forest Grove and Hillsboro's Upper System.

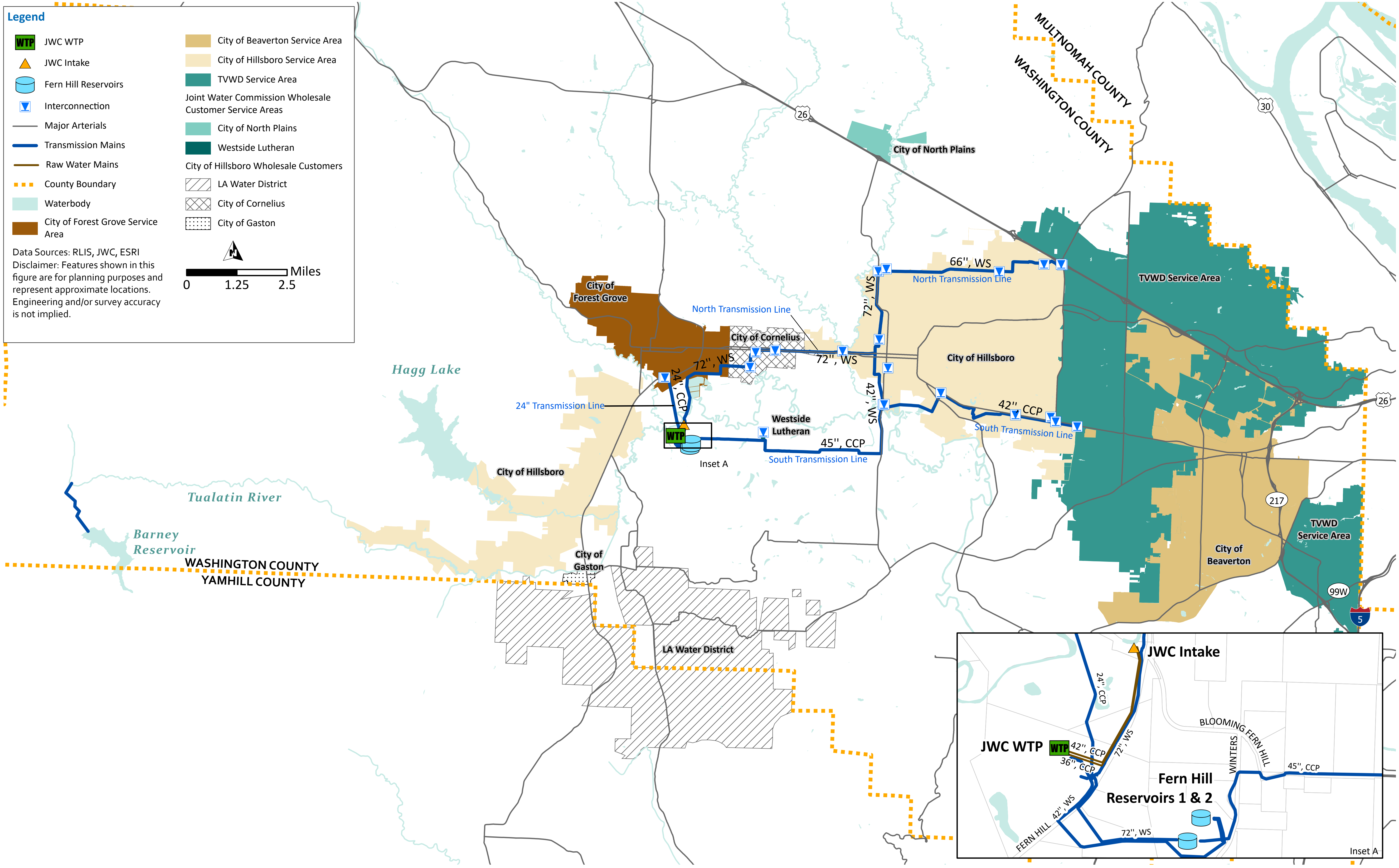


Figure ES.2 JWC Existing System

ES.3 Planning Considerations

In critically evaluating the existing water system and setting expectations for operational and capital improvements, JWC upholds policies and criteria organized into the following categories:

- Ownership guidelines for the four member agencies.
- Planning criteria related to operating JWC's water system.
- Production strategy that guides system operations and water production.
- Sustainability goals that coordinate the member agencies' conservation efforts.
- Maintenance policies related to asset and information security.

JWC also establishes seismic level of service goals that guide the design standards for water system infrastructure, which must recover rapidly and effectively following an earthquake. JWC strives to restore system operations to provide winter average day demand within 1 day after a magnitude 9.0 Cascadia Subduction Zone earthquake and 7 to 14 weeks after an earthquake with a 2,475-year return period.

ES.4 Water Quality and Regulatory Framework

JWC maintains a robust raw and finished water quality sampling program, which confirms that contaminants in their finished water continue to fall below maximum contaminant levels (MCLs) set by the U.S Environmental Protection Agency (EPA) and the OHA in Oregon Administrative Rule (OAR), Chapter 333, Division 061.

This Plan also reviewed contaminants of emerging concern and established water quality goals above and beyond regulatory requirements.

ES.5 Water Demand Projections

On average, the JWC produces and distributes an average day demand of 31 mgd of treated water to their member agencies. As shown in Figure ES.3, the JWC WTP's production has historically fluctuated throughout the years, with daily peak production typically falling below 30 mgd during the winter months and above 50 mgd in the summer months. Production reached its historical peak in July 2015 with a maximum daily production of 65 mgd.

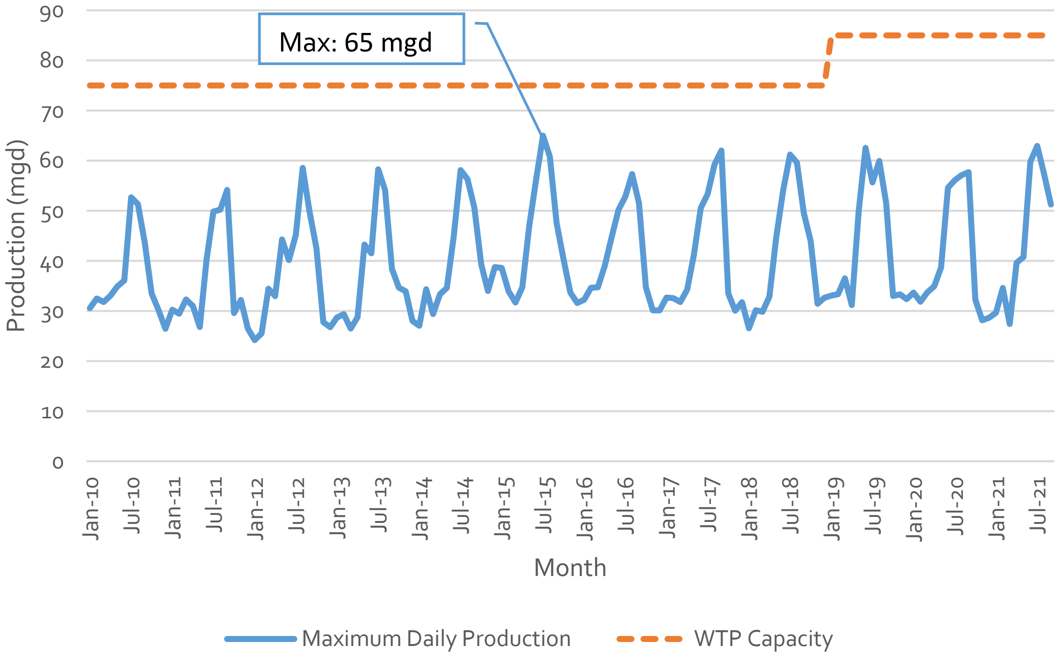


Figure ES.3 Maximum Daily JWC WTP Production by Month

To project JWC's water demand through the next 20 years, each member agency and wholesale customer's respective demand projections were compiled. Figure ES.4 shows that the JWC WTP's current capacity of 85 mgd is expected to sufficiently meet the demands of all members through this Master Plan's planning horizon.

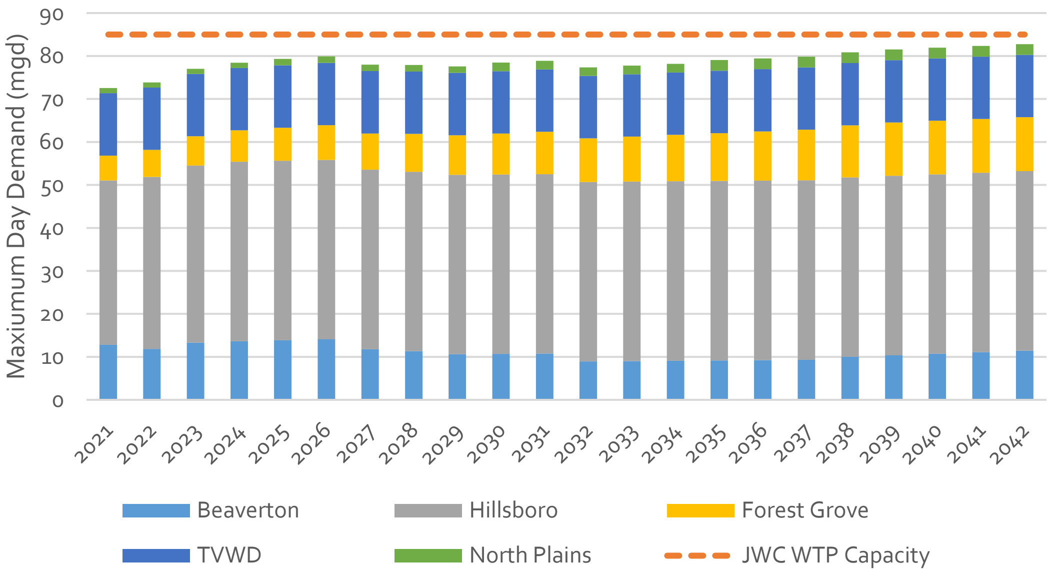


Figure ES.4 JWC WTP Partner Demand Projections through 2042

Still, certain members may require more water than their current allocations before 2042. For instance, Forest Grove is anticipated to exceed their 10-mgd allocation by 2032 but only by 1 to 3 mgd until 2042. Rather than expanding the capacity of the intake or WTP, JWC plans to meet this additional demand by partners with excess capacity leasing unused capacity to partners who need this supply, within the terms of the intergovernmental agreement.

ES.6 Evaluations of Key Infrastructure

To prepare a targeted and cost-efficient capital improvements program, this Plan critically evaluated key infrastructure within the water system, namely the SHPP intake, JWC WTP, and transmission system. Identified needs were developed into capital improvement projects that will be implemented over the next 20 years.

ES.6.1 Intake Evaluation

As it exists, the SHPP is not seismically resilient and its current total 88-mgd capacity is anticipated to be surpassed by year 2050 by JWC's maximum day water demand. Furthermore, the SHPP's firm capacity of 67 mgd is less than the 88-mgd firm capacity required within the 20-year planning horizon.

To reliably secure water supplies for their member agencies, JWC performed economic and non-economic evaluations of the following three alternatives to understand their feasibility and long-term benefit to intake operations:

- **Alternative 1**: Develop a new, seismically resilient intake on Property 1, which borders the east bank of the Tualatin River and is located just downstream and to the north of the existing SHPP.
- **Alternatives 2a (with a fish ladder and concrete weir) and 2b (without a fish ladder and concrete weir)**: Develop a new, seismically resilient intake on Property 2, an agricultural site that shares a boundary along the JWC WTP's western edge. This alternative may require a fish ladder and concrete weir structure.
- **Alternative 3**: Upgrade the capacity and seismic resilience of the existing SHPP.

The selected alternative must provide an initial firm capacity of 88 mgd, an interim firm capacity of 105 mgd, and a build-out firm capacity of 125 mgd under all river flow conditions, 44 mgd of which must be seismically resilient. It must also comply with all relevant environmental, land use, and floodplain permitting.

ES.6.1.1 Economic Evaluation

The economic evaluation compared alternative 1, 2a and 2b's Association for the Advancement of Cost Engineering (AACE) Class 5 estimate, which has an expected accuracy range of minus 20 percent to minus 50 percent on the low end and plus 30 to plus 100 percent on the high end. Due to the lack of definition for Alternative 3, an AACE Class 10 cost estimate is provided.

Tables ES.1 and ES.2 compare the costs of the three alternatives. As can be seen, Alternative 1 represents the highest cost, with Alternative 3 closely following. Alternative 2a costs almost half of Alternative 1, which still costs more than Alternative 2b.

Table ES.1 Conceptual-Level Cost Comparison of Tualatin River Intake Alternatives

No.	Description	Estimated Conceptual Project Cost ($M)
1	Property 1: • 88-mgd intake and pump station. • Concrete weir structure with gates. • Five-step fish ladder. • 3500-foot x 42-inch-diameter RWPs.	$49M
2A	Property 2: • 88-mgd intake and pump station. • Concrete weir structure with gates. • Three-step fish ladder. • 1700-foot x 42-inch-diameter RWPs.	$34M
2B	Property 2: • 88-mgd intake and pump station. • No concrete weir structure or fish ladder. • 1700-foot x 42-inch-diameter RWPs.	$25M

Note:
Abbreviations: RWP - raw water pipeline; $M - million dollars.

Table ES.2 Cost Estimate to Upgrade SHPP

No.	Alternative Name	Estimated Conceptual Project Cost ($M)
3	Upgrade the SHPP: 88 mgd	$48

ES.6.1.2 Non-economic Evaluation

The non-economic evaluation assessed the alternatives' qualitative benefits, which are represented by the following criteria:

- Sufficient water quantity.
- Resilience and reliability.
- Simplicity of land acquisition.
- Minimized impacts of environmental and land use permitting.
- Constructability.
- Ease of operations and maintenance (O&M).

The JWC WTP's staff and the member agencies weighted each criterion on a scale of 1 (i.e., least importance) to 5 (i.e., most important) and then scored each alternative on a scale of 1 (i.e., worst in meeting criterion) to 5 (i.e., best in meeting criterion). To determine a score per criterion and weighted overall score per alternative, assigned scores were multiplied by each criterion's corresponding weight factor.

Figure ES.5 summarizes the results of this collaborative evaluation process.

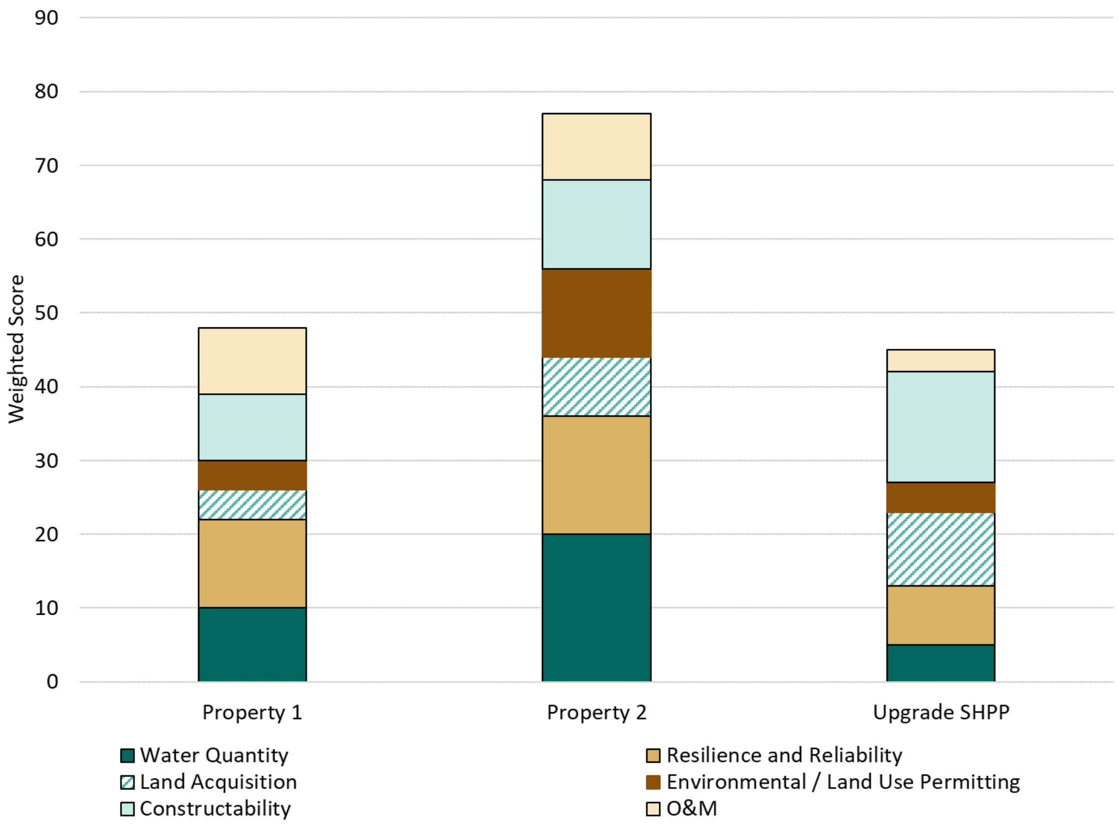


Figure ES.5 Intake Alternatives Non-Economic Scores

The figure shows that Alternative 2 fulfills the most important criteria of water quantity, resilience and reliability, and environmental and land use permitting. More specifically, the depth of the river at Property 2 helps meet JWC's intake capacity goals with the smallest footprint and likely without a concrete weir and fish ladder. And, because it requires the shortest raw water pipeline, this alternative is also considered the most resilient.

ES.6.1.3 Final Recommendation

This alternatives evaluation recommends that JWC construct their own intake facility on the Tualatin River and employ the SHPP to the end of its useful life as a redundant supply source. JWC's ownership of a new property and intake will offer them more control over the resilience and reliability of their infrastructure and supplies.

Of the three alternatives evaluated, the overall best long-term intake solution is the construction of an intake facility on Property 2. JWC is recommended to pursue a lot line adjustment with the owners of Property 2 to append approximately 6 acres to the JWC WTP property.

ES.6.2 JWC Water Treatment Plant

The *2018 Water Treatment Facility Plan* (2018 Facility Plan) established a robust road map for future improvements and expansion of the JWC WTP. For this Master Plan, the primary focus was an alternatives evaluation the replacement of the current disinfection system. The JWC WTP currently utilizes chlorine gas for disinfection and other treatment purposes; however, this highly toxic chemical poses safety challenges, has ongoing O&M issues associated with the chlorine scrubber system, and lacks reliable suppliers in the Pacific Northwest.

This Plan screened disinfection alternatives, developed design criteria and conceptual layouts, and completed economic and non-economic evaluations on the following four alternatives to replace the existing chlorine gas system:

- **Bulk sodium hypochlorite (bulk hypochlorite):** Construct chemical storage and metering facilities to use liquid bulk sodium hypochlorite for disinfection, with all sodium hypochlorite being delivered to the WTP.
- **Full on-site hypochlorite generation (OSHG) with full bulk sodium hypochlorite backup (full OSHG):** Construct an OSHG system to generate dilute liquid sodium hypochlorite on-site from delivered salt and meet disinfection requirements across all plant flow ranges.
- **Bulk sodium hypochlorite with backup OSHG (bulk hypochlorite with backup OSHG):** Construct chemical storage and metering facilities sized to use bulk liquid sodium hypochlorite for disinfection across all plant flow ranges, and build a small, backup OSHG system sized only to meet wintertime average day demands at average chemical doses.
- **Bulk sodium hypochlorite with provisions for future conversion to full OSHG (bulk hypochlorite future OSHG):** Construct chemical storage and metering facilities to use bulk liquid sodium hypochlorite, with all sodium hypochlorite delivered to the WTP, for the near-term. Design the new chemical facilities with adequate space and provisions to accommodate installation of an OSHG system in the future.

The selected alternative must accommodate the JWC WTP's current capacity of 85 mgd. For the evaluations in this master plan, all alternatives were sized to accommodate an ultimate capacity of 125 mgd at build-out.

ES.6.2.1 Economic Evaluation

The economic evaluation consisted of development of the following cost estimates for each alternative:

- Total construction project costs which were mostly focused on major facilities and equipment.
- Annual fixed and variable O&M costs, including chemical usage, power usage, and equipment maintenance.
- Net present value (NPV) costs for 20 years of operation (i.e., 2022 to 2042) to account for differences in annual operating costs among the alternatives.

Figure ES.6 summarizes the results of the alternatives' comparative cost evaluation. As can be seen, bulk hypochlorite and bulk hypochlorite future OSHG have the lowest capital costs and 20-year NPV even with the high annual O&M costs for chemical delivery. Bulk with provisions for future OSHG has a $0.2 million higher capital cost and 20-year NPV than bulk hypochlorite that is driven by the larger building footprint to accommodate full OSHG in the future.

Due to the need for OSHG equipment and an overall larger footprint, full OSHG's capital costs are nearly 60 percent higher than the bulk hypochlorite alternatives, and the 50 percent reduction in O&M costs relative to bulk hypochlorite is not substantial enough to offset the difference in capital costs for the 20-year NPV.

Bulk hypochlorite with backup OSHG has capital costs between bulk hypochlorite and full OSHG, with cost savings accrued from the smaller size of the generation system. Annual O&M costs are comparable to bulk hypochlorite and assume 11 months of operation with bulk hypochlorite, but savings earned from operating the OSHG system only one month per year are offset by the increased equipment maintenance costs. As such, this alternative's high annual operating costs and higher capital cost result in the highest 20-year NPV cost.

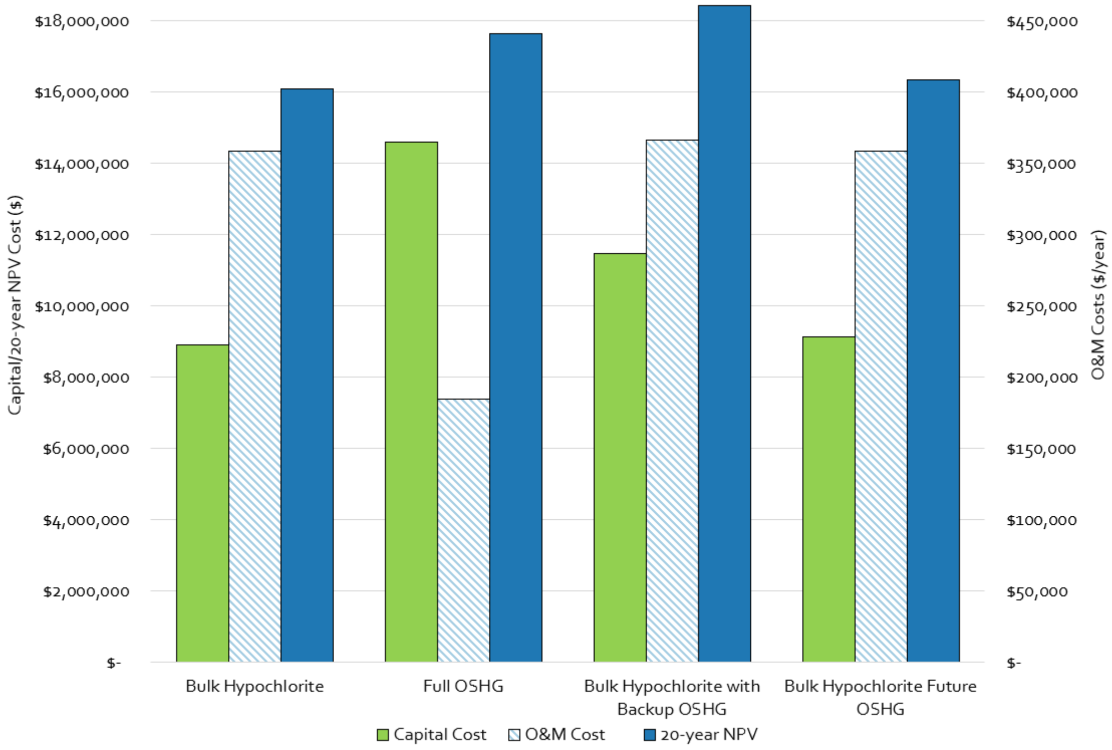


Figure ES.6 Summary of Capital, O&M, and NPV Costs for Each Alternative

ES.6.2.2 Non-economic Evaluation

The non-economic evaluation assessed the alternatives' qualitative benefits against the same six criteria employed for the SHPP's non-economic evaluation. JWC WTP staff members and JWC partners weighted each criterion on a scale of 1 (i.e., least importance) to 5 (i.e., most important) and then scored each alternative on a scale of 1 (i.e., worst in meeting criterion) to 5 (i.e., best in meeting criterion). Assigned scores were multiplied by each criterion's assigned weight factor to determine a score per criterion and then totaled to generate a weighted overall score for each alternative.

Figure ES.7 summarizes the results of this process. As shown, full OSHG scores the highest, with notable leads in the most important criteria, safety and resilience and reliability. OSHG poses few health and safety risks to WTP staff and the surrounding community and does not rely on corrosive bulk hypochlorite. Instead, it utilizes salt, which is a non-hazardous chemical that can be more easily supplied even after a local supply outage than bulk hypochlorite. Finally, the full OSHG system, which can also act as a storage tank for bulk deliveries of backup supplies of sodium hypochlorite during peak demand, is wholly redundant.

The bulk hypochlorite with future OSHG scores slightly higher than the bulk hypochlorite alternative for Resilience & Reliability because planning for the future OSHG sets the stage for JWC to convert to a more resilient source of sodium hypochlorite in the future.

All alternatives scored similarly in terms of water quality and environmental and land use permitting. Bulk hypochlorite scores the highest for constructability and O&M categories for requiring the smallest footprint and least amount of equipment to maintain while the remaining two alternatives score similarly in these areas. However, bulk hypochlorite with backup OSHG does score the lowest for O&M because it will be the most complex to operate and maintain.

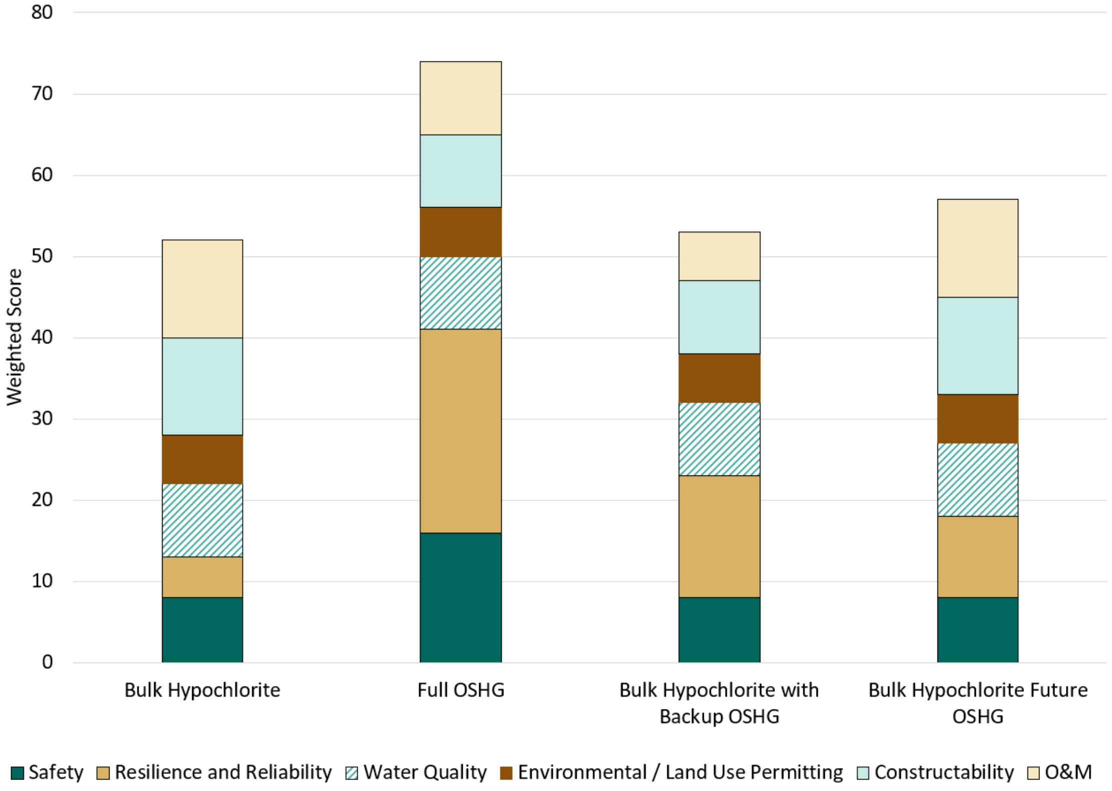


Figure ES.7 Disinfection Alternatives Non-Economic Scores

ES.6.2.3 Final Recommendations

The evaluations, paired with extensive discussions with JWC WTP staff and the JWC partners, determined that, while full OSHG does come with many substantial benefits in the long term, current bidding and construction challenges and the high upfront capital cost prevent it from being immediately implemented. As such, JWC decided to select the bulk hypochlorite with provisions for future conversion to OSHG.

Figure ES.8 shows a conceptual layout for the selected alternative. The total estimated capital cost is $9.1M, which does not include capital costs associated with future OSHG equipment or salt storage.

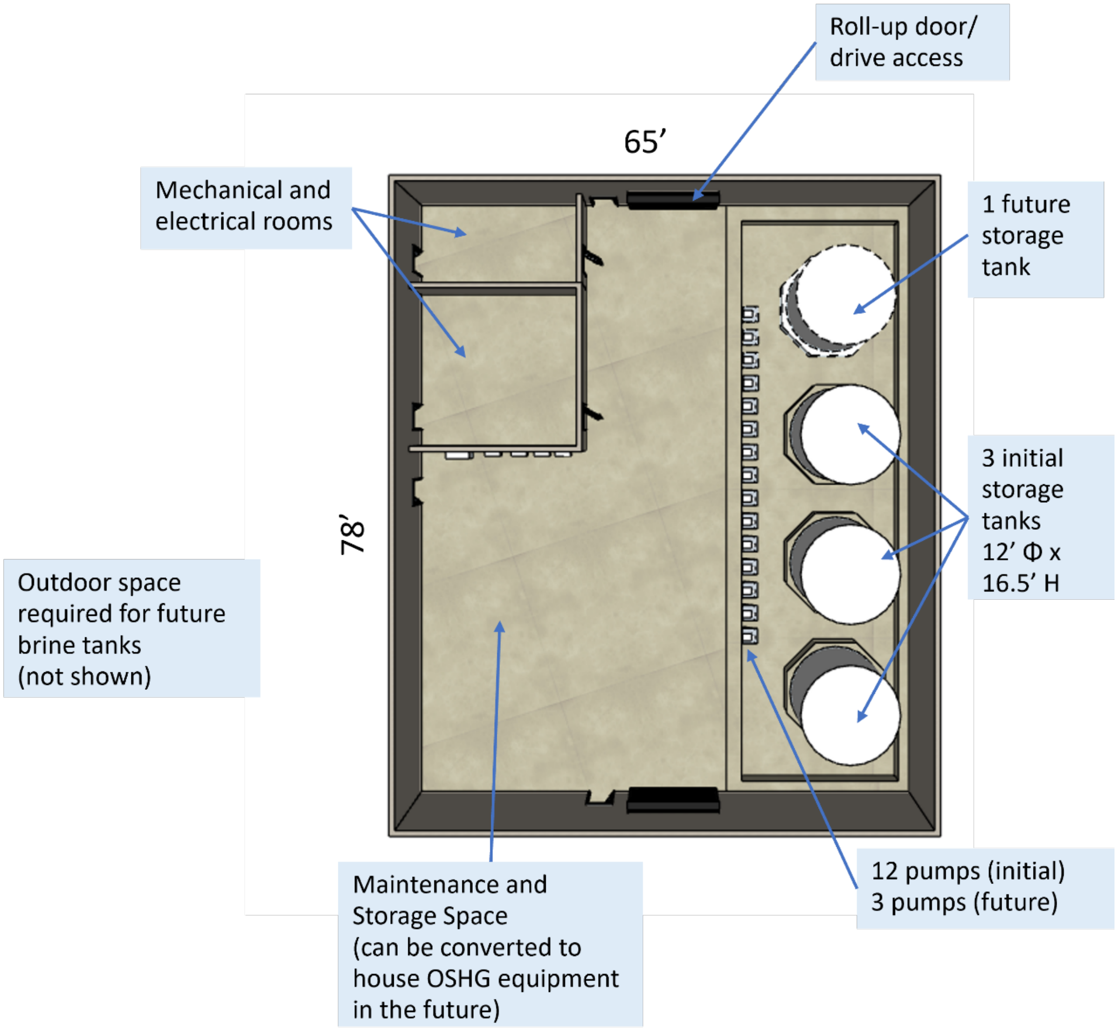


Figure ES.8 Layout of Bulk Hypochlorite Disinfection to be Expanded to Full OSHG in the Future

In addition to the major replacement of the new disinfection process, WTP improvements identified in this Master Plan were combined with recommended improvements from past studies, particularly the 2018 Facility Plan, to facilitate development of a capital improvement program for JWC. Improvement projects were grouped into projects that address O&M challenges, projects that meet seismic and resilience requirements, and projects that target future expansion needs beyond the current 85 mgd capacity.

During CIP development, improvement projects were sequenced and prioritized, which differed from what was established in the 2018 Facility Plan. A siting study was conducted to develop an updated site layout and vision for the JWC WTP that accounts for all current CIP projects and follows the timing and prioritization identified in the CIP. Three site layouts were developed and a preferred site layout was selected after consultation with JWC staff and the JWC partners. The preferred site layout aligns with the CIP developed in this Master Plan, with several adjustments that can be considered during future project implementation without affecting the overall vision for the JWC WTP. Details on the siting study evaluation are provided in Appendix O.

ES.7 Transmission System

To confirm adequate capacities for present and future demand, JWC's transmission system, including the Fern Hill reservoirs and the STL, NTL, and FGHB, were carefully analyzed against the JWC's storage, minimum hydraulic grade line, and maximum transmission main velocity planning criteria. The hydraulic model of JWC's transmission system was used to evaluate the system against the planning criteria.

Results showed that the reservoirs and pipelines have sufficient capacity for the 20-year planning horizon. The firm capacity of JWC's finished water pumps is below that of the JWC WTP. To increase the firm capacity, a new pump is recommended to be installed at Finished Water Pump Station 2.

The hydraulic model was also used to simulate power failure events under a surge analysis of the water system, with a focus placed on the two finished water pump stations, the Fern Hill reservoirs, the surge vessel near the pump stations, the surge bladder, and air valves. Results indicated that several assets compromise the water system's surge protection, which can be addressed through capital improvement efforts.

Following the review of past and present evaluations and their recommendations, a total of 12 projects that address the transmission system's pump station capacity, cathodic protection, pipeline infrastructure needs, pipeline condition, surge protection, and seismic resiliency are recommended to be added to this Plan's CIP.

ES.8 Capital Improvement Program

The recommended projects in this Plan were combined into a comprehensive CIP. The CIP consists of cost estimates and prioritization for each recommended improvement project and serves as a guide for future planning, budgeting, and financial evaluations.

This Plan encompasses a 20-year planning horizon and includes a 20-year CIP through FY 2042-43, which was subdivided into the following phases:

- Phase 1: FY 2023-24 through FY 2027-28.
- Phase 2: FY 2028-29 through FY 2032-33.
- Phase 3: FY 2033-34 through FY 2042-43.

This Plan also includes a 40-year resiliency CIP, which includes projects beyond the 20-year planning horizon in Phase 4: FY 2043-44 through FY 2062-63.

The JWC classifies capital projects by project category. Table ES.3 presents a summary CIP by project category and CIP phase.

Table ES.3 CIP Summary by Project Category

Project Category	Total CIP Cost	Phase 1 (FY2024-28)	Phase 2 (FY2029-33)	Phase 3 (FY2034-43)	Phase 4 (FY2044-63)
Total Cost	**$277,845,000**	**$23,490,000**	**$18,141,000**	**$92,805,000**	**$140,983,000**
Land	$1,000,000	$1,000,000	$-	$-	$-
Treatment Facilities	$203,923,000	$17,245,000	$3,190,000	$72,505,000	$110,983,000
Transmission Lines	$67,770,000	$4,376,000	$13,094,000	$20,300,000	$30,000,000
Water Rights	$200,000	$200,000	$-	$-	$-
SCADA	$4,952,000	$3,095,000	$1,857,000	$-	$-
Annual Cost	-	$5,183,000	**$3,628,000**	**$9,281,000**	**$7,049,000**

Each CIP project was assigned an ownership allocation based on the infrastructure involved in the project. Tables ES.4 and ES.5 summarize the total CIP cost by JWC partner and phase and annual CIP cost by JWC partner and phase, respectively.

Table ES.4 Total CIP Summary by JWC Partner

JWC Partner	Total CIP Cost	Phase 1 (FY2024-28)	Phase 2 (FY2029-33)	Phase 3 (FY2034-43)	Phase 4 (FY2044-63)
Total Cost	**$277,845,000**	**$25,916,000**	**$18,141,000**	**$92,805,000**	**$140,983,000**
Hillsboro	$136,056,000	$12,309,000	$8,911,000	$45,585,000	$69,251,000
Forest Grove	$32,640,000	$3,048,000	$2,133,000	$10,879,000	$16,580,000
TVWD	$61,260,000	$5,743,000	$4,002,000	$20,414,000	$31,101,000
Beaverton	$47,890,000	$4,816,000	$3,095,000	$15,927,000	$24,052,000

Table ES.5 Annual CIP Costs by JWC Partner

JWC Partner	Phase 1 (FY2024-28)	Phase 2 (FY2029-33)	Phase 3 (FY2034-43)	Phase 4 (FY2044-63)
Annual Cost	**$5,183,000**	**$3,628,000**	**$9,281,000**	**$7,049,000**
Hillsboro	$2,462,000	$1,782,000	$4,558,000	$3,463,000
Forest Grove	$610,000	$427,000	$1,088,000	$829,000
TVWD	$1,149,000	$800,000	$2,041,000	$1,555,000
Beaverton	$963,000	$619,000	$1,593,000	$1,203,000

After the CIP was established, each project was evaluated for funding eligibility. Table ES.6 lists the projects most eligible for funding and the potential funding sources. Details on each of these funding sources and how they are applicable to JWC projects can be found in Appendix N.

Table ES.6 Summary of CIP Projects and Funding Opportunities

No.	Project Name	Timing	Low-Interest Loan: EPA; WIFIA	Low-Interest Loan: Oregon IFA SDWRLF	Legislative: Community Project Funding	FEMA: BRIC	FEMA: FMA	FEMA: HMGP	USBOR: Water & Energy Efficiency Grants	USBOR: Small-Scale Water Efficiency Projects	DOE: Energy Efficiency and Conservation Block Grant Program	OWRD: Water Project Grants & Loans	OEM: SPIRE
S-01	SCADA Program	2024 to 2031	√	√	√				√		√	√	
T-24	15,000-Gallon Diesel Fuel Tank(1)	2024	√	√	√								√
T-03	Hazard Mitigation Study	2024 to 2026	√	√	√	√		√					
T-18	Replace Chlorine Gas System	2024 to 2026	√	√	√			√					
T-05	Initiate Study with USBOR for SHPP(2)	2026 to 2027											
D-09	Beaverton and TVWD Billing Meters	2027	√	√						√	√		
D-03, D-08	Seismic Valves on N-S Intertie and Fern Hill Force Mains/Replace/Improve Transmission Lines in Critical High-Hazard Areas (20-year CIP)(3)	2029 to 2043	√	√	√	√		√					
T-06	Hazard Mitigation Implementation	2029 to 2033	√	√	√			√					
T-07, T-21	Replace Rapid Mix Facility/ Replace O&M Building(4)	2034 to 2036	√	√	√			√			√		
T-19	Install OSHG Equipment into Hypo Facility(5)	2037 to 2039	√	√	√	√		√					
T-20	Construct New Chemical Building(6)	2037 to 2039	√	√	√	√		√			√		
T-04	New Intake or SHPP Upgrade	Beyond 2044	√	√	√	√	√	√					

Notes:

Abbreviations: BRIC – Building Resilient Infrastructure and Communities; DOE – Department of Energy; FEMA – Federal Emergency Management Act; FMA – Flood Mitigation Assistance; HMGP – Hazard Mitigation Grant Program; IFA – Business Oregon Infrastructure Finance Authority; OEM – Oregon Department of Energy Management; OWRD – Oregon Water Resources Department; SDWRLF – Safe Drinking Water State Revolving Loan Fund; SPIRE – State Preparedness and Incident Response Equipment; WIFIA – Water Infrastructure Finance Innovation Act.

(1) Project cost less than JWC funding threshold of $1 million, but favorable funding opportunity is available.

(2) JWC should consider approaching USBOR for funding assistance with study.

(3) Due to length of project, D-08 could be phased and combined with D-03 to maximize available funding opportunities.

(4) Because projects are so closely aligned, projects may be combined to maximize funding opportunity.

(5) Project may be funded alone, but JWC should consider combining project with T-20 to maximize funding opportunity.

(6) Project may be funded alone, but JWC should consider combining project with T-19 to maximize funding opportunity.

Chapter 1

INTRODUCTION

1.1 Introduction

Established in 1976, the Joint Water Commission (JWC) is an intergovernmental water supply agency formed under an Oregon Revised Statute (ORS) 190 agreement between the Cities of Hillsboro (Hillsboro), Forest Grove (Forest Grove), Beaverton (Beaverton), and the Tualatin Valley Water District (TVWD). With ownership shared between the four remaining member agencies, JWC acts as the primary drinking water supplier to approximately 450,000 customers in the growing communities of Washington County, Oregon.

To remain ahead of customer demand and uphold a safe and reliable system, JWC maintains a living capital improvement program (CIP) within a water master plan. JWC updates this master plan regularly to diagnose the current condition, capacity, and resilience of their water system and establish a 20-year roadmap for successful operations and capital improvements.

This *2023 Master Plan* (Plan) updates its preceding iteration, *JWC Capital Improvement Master Planning Services: 2009 Master Plan*, and prepares a robust CIP that encompasses a 20-year planning horizon from fiscal year (FY) 2023/24 through FY 2042/43. Resilience projects are spread out over a 40-year planning horizon from FY 2023/24 through FY 2062/63. Projects within the CIP were strategically identified as part of the analyses performed during development of this Plan and carried over from previous studies.

In accordance with Chapter 333-061 of the Oregon Administrative Rules, Oregon Health Authority (OHA) requirements this Plan:

- Describes the existing JWC system and its infrastructure.
- Establishes the policies and criteria and level of service (LOS) goals against which the system will be evaluated.
- Presents demand projections for all JWC member agencies to project future water supply requirements.
- Evaluates the JWC Spring Hill Pumping Plant (SHPP), Water Treatment Plant (WTP) and transmission system to identify deficiencies and recommend improvements.
- Develops a seismic risk assessment and mitigation plan for the transmission system.
- Develops a capital improvement program outlining recommended improvements to the JWC system with cost estimates and schedules for project completion for the next 20 years. Additionally, seismic resilience projects are included in a 40-year CIP.

1.1.1 Objectives

The following key master planning objectives were used to guide development of this Plan:

- **Objective No. 1**: Develop a vision for the future of JWC that continues the commission's history of successful collaboration, garnering consensus among all JWC members.

- **Objective No. 2**: Compile relevant planning information from existing studies, verify the recommendations are consistent with JWC's level of service goals, and identify remaining questions or policy gaps. Fill the gaps identified to lay a foundation for the next 20 years of successful JWC operations and capital improvements.
- **Objective No. 3**: Evaluate the capacity and resilience of the water system and recommend projects required to meet JWC's level of service goals. Analyze various alternatives against established criteria in order to select the best option to address each challenge identified.
- **Objective No. 4**: Develop a comprehensive and achievable capital improvement program for the next 20 years. Develop a seismic resiliency improvement plan for the next 40 years. Establish criteria to prioritize projects.

1.2 Overview of JWC

JWC is a collective water supply agency formed under an ORS 190 agreement between the cities of Hillsboro, Forest Grove, Beaverton, and the TVWD. Through its member agencies, JWC provides the primary potable water supply to approximately 450,000 customers. The ownership of facilities among the group includes raw water impoundment in both Barney Reservoir and Scoggins Reservoir, an 85 million gallon per day (mgd) water treatment plant located along the Tualatin River, two 20 million gallon (MG) finished water reservoirs located at the top of Fern Hill, and three transmission lines that deliver finished water to the JWC members.

Chapter 2 describes the components of the existing system and WTP in further detail.

The JWC system is officially designated in OHA records as "Joint Water Commission" with public water system identification number 4100379 and is classified as wholesale community water system.

JWC is run by a board of 12 Commissioners, three Commissioners from each member agency. Day-to-day operations are overseen by three committees:

1. Management Committee.
2. Operations Committee.
3. Events and Education Committee.

At the time of the development of this Plan, the City of Hillsboro was serving as the Managing Agency for JWC and providing management and staff for all JWC system operations.

1.3 Approval Process

The JWC Board of Commissioners authorized the preparation of this document in accordance with all JWC policies and procedures, and all applicable federal, State, and Washington County rules and regulations.

Management and Operations Committee members were updated on the status of this Plan throughout the development process. The Plan was submitted to OHA for approval as part of the agency review process. In addition to OHA, the draft Plan was sent to all Commissioners and JWC member agencies for their review. Appendix B includes the review comments and responses.

Appendix A presents the adopting resolution for the final Plan.

1.4 Previous Studies and Projects

The following list highlights key projects and studies completed for JWC since the 2009 Master Plan that were reviewed during development of this Plan:

- 2020 Water Treatment Plant Expansion to 85 mgd plans and drawings.
- *2020 Water Management and Conservation Plan*.
- *2020 JWC Operations Manual*.
- *2020 SCADA* [supervisory control and data acquisition] *Master Plan.*
- *2020 Cathodic Protection Study.020 AWIA Risk and Resilience Assessment.*
- *2019 Surge Analysis Report*.
- *2018 Water Treatment Plant Facility Plan* (2018 Facility Plan).
- *2019 Hillsboro Water Master Plan.*
- *2019 Beaverton Water System Master Plan*.
- *2018 TVWD Water Master Plan Update.*
- *2015 Capital Improvement Program Update*.
- *2012 Electrical Coordination Study and System Assessment*.
- *2010 Forest Grove Water Master Plan*.
- 2009 Master Plan.
- 2008 Seismic Hazard Mitigation Study.
- *Technical Memorandum No. 1: Capital Improvement Program Update*

1.5 Intergovernmental Agreements

The JWC Water Service Agreement, last amended January 10, 2014, defines the agreement between the four JWC partners, Hillsboro, Forest Grove, and Beaverton, and the TVWD. Amongst other conditions and requirements, this document describes the powers and management of the JWC, shared ownership of JWC facilities, use of the JWC system, and payment for construction and operation of JWC facilities. A copy of the complete agreement is provided in Appendix C.

Appendix C also includes the current intergovernmental agreement between JWC and the North Plains which remains in effect until January 1, 2025, and is automatically renewed for three successive five-year renewal periods unless either party provides written notice to re-negotiate or terminate the agreement. Under this agreement, JWC agrees to provide all of North Plains municipal water supply at agreed upon maximum daily demands. North Plains is to maintain back-up well supplies and maintain its own facilities.

1.6 Report Organization

This Plan contains eight chapters and additional appendices that provide supporting documentation and analysis. Plan chapters are briefly described below:

- **Chapter 1 – Introduction:** This chapter summarizes the key objectives and organization of this Plan.
- **Chapter 2 – Existing System:** This chapter describes the existing JWC facilities.
- **Chapter 3 – Planning Considerations:** This chapter summarizes the water system policies and planning criteria that govern administration, operation, and management of the water system plan as well as hazard resilience LOS goals.

- **Chapter 4 – Water Quality and Regulatory Framework:** This chapter compares historical water quality against existing and future regulatory requirements, identifies contaminants of emerging concern, and establishes water quality goals.
- **Chapter 5 – Water Demand Forecast and Supply Strategy:** This chapter develops a water demand forecast for all JWC partners and evaluates supply needs to meet partner supply needs for the next 20 years.
- **Chapter 6 – Intake Evaluation:** This chapter presents an alternatives analysis for a future resilient intake on the Tualatin River and discusses the benefits and challenges of establishing a raw water pipeline from Scoggins Dam to the WTP.
- **Chapter 7 – Water Treatment Plant Evaluation:** This chapter presents an alternatives analysis for replacement of JWC's gaseous chlorine disinfection system and establishes a water treatment plant improvements plan founded on recommendations from previous studies.
- **Chapter 8 – Transmission System Evaluation:** This chapter discusses hydraulic evaluations of the JWC transmission system and recommends improvements to mitigate identified deficiencies. It also presents a surge analysis of JWC's transmission mains and summarizes recommendations from the cathodic protection plan, seismic evaluation, and pipeline condition analysis.
- **Chapter 9 – Capital Improvement Program:** This chapter consolidates and prioritizes recommended improvements into a comprehensive capital improvement program for the next 20 years. The CIP also includes seismic resilience projects to be completed over the next 40 years.

1.7 Acknowledgments

This Plan was developed as a collaborative effort between JWC, Carollo Engineers, Inc., SEFT Consulting Group, GSI Water Solutions, Inc., McMillen Jacobs Associates, David Evans and Associates, and R&W Engineering, Inc. We'd particularly like to thank the following people from the JWC partner agencies for their contribution, participation in numerous workshops, and review of draft chapters and memos throughout the development of this Plan:

- Chris Wilson, City of Hillsboro.
- David Winship, City of Beaverton.
- Derek Robbins, City of Forest Grove.
- Eric Hielema, City of Hillsboro.
- Jessica Dorsey, City of Hillsboro.
- Kevin Meeuwsen, City of Hillsboro.
- Lee Lindsey, City of Hillsboro.
- Mellisa Franklin, City of Hillsboro.
- Negar Niakan, City of Hillsboro.
- Nesh Mucibabic, City of Hillsboro.
- Nicholas Augustus, TVWD.
- Rachel Moore, City of Hillsboro.
- Sarah Bruce, City of Hillsboro.
- Sarah Honious, City of Hillsboro.
- Sophia Hobet, City of Hillsboro.
- Zac Bertz, City of Hillsboro.

Chapter 2

EXISTING SYSTEM

2.1 Introduction

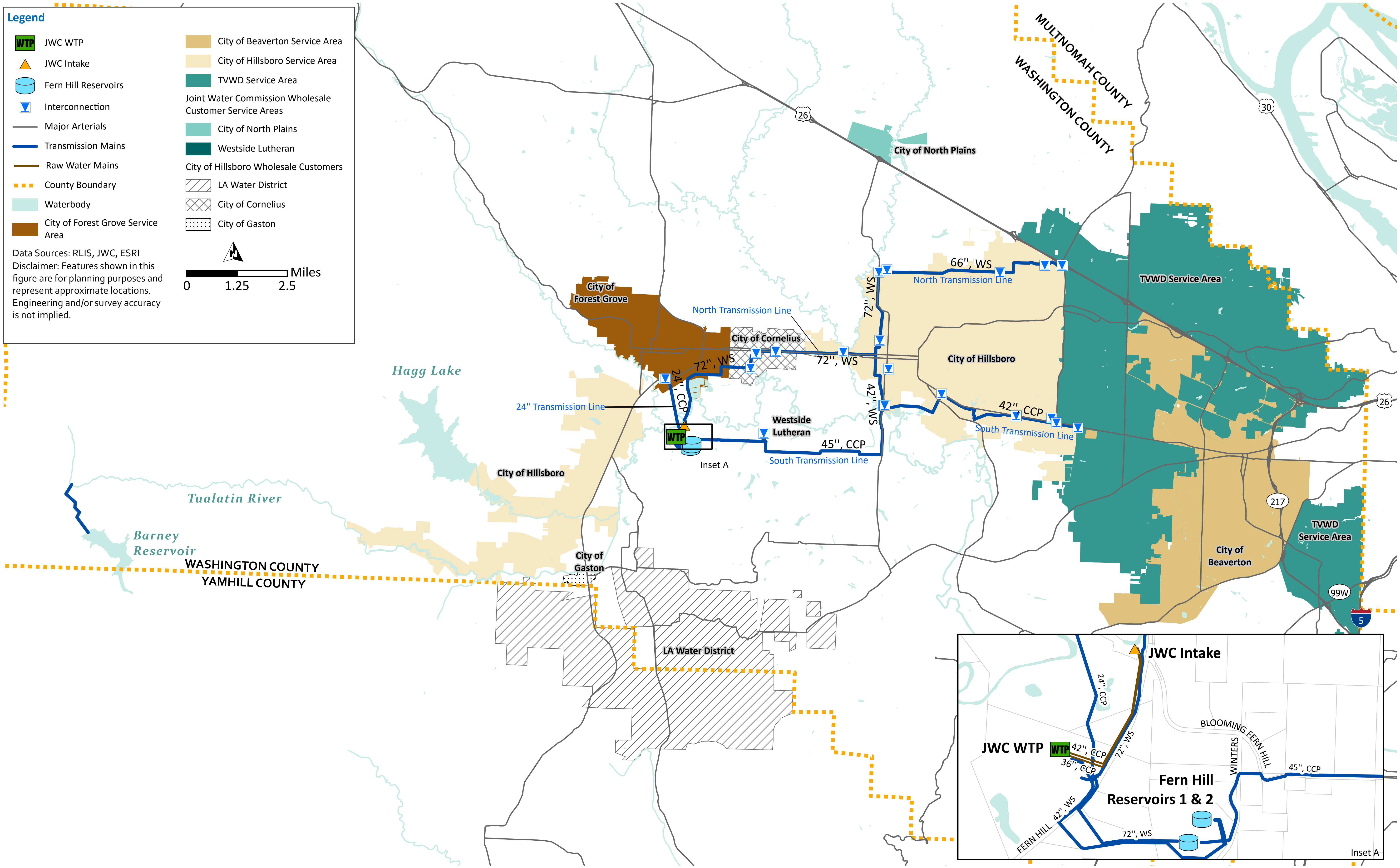
The Joint Water Commission's (JWC) water system serves approximately 450,000 customers in Washington County, Oregon. The JWC system consists of the following primary assets:

- The Spring Hill Pumping Plant (SHPP) intake on the Tualatin River.
- The JWC Water Treatment Plant (WTP).
- Two Fern Hill storage reservoirs.
- Three transmission mains with interconnections to serve the four JWC member agencies—the City of Forest Grove (Forest Grove), City of Hillsboro (Hillsboro), Tualatin Valley Water District (TVWD), and City of Beaverton (Beaverton)—and two wholesale customers, the City of North Plains (North Plains) and Westside Lutheran School.
- An interconnection in Hillsboro's Upper System, through which Hillsboro provides wholesale water to the City of Gaston and L.A. Water Cooperative. Hillsboro also provides wholesale water to the City of Cornelius (Cornelius) through interconnections off the North Transmission Line.

Figure 2.1 shows a map of the JWC water system's service area.

JWC's water supply comes from two sources: The Tualatin River and the Middle Fork of the Trask River's North Fork. A 36-inch pipeline that is 6,500 feet in length cuts across the narrow Oregon Coast Range divide between the two watersheds and allows JWC to release stored water from the Barney Reservoir within the Trask River watershed into the Tualatin River watershed where it is diverted at the SHPP intake.

Pumped raw water is treated at the 85-million-gallon-per-day (mgd) JWC WTP, a conventional media-filtration plant that features coagulation, flocculation and sedimentation, filtration, and disinfection processes. Finished water is conveyed to JWC customers via three main transmission lines: the North Transmission Line (NTL), the South Transmission Line (STL), and the Forest Grove (FG) transmission main. The NTL serves Hillsboro, North Plains, Cornelius, and TVWD, while the STL serves Hillsboro, TVWD, and Beaverton. The FG line feeds Forest Grove as well as Hillsboro's Upper System. Finished water is first fed to NTL and FG lines, with excess water being fed to the Fern Hill Reservoirs. From these reservoirs, water is fed to customers via the STL.



Data Sources: RLIS, JWC, ESRI
Disclaimer: Features shown in this figure are for planning purposes and represent approximate locations. Engineering and/or survey accuracy is not implied.

Figure 2.1 JWC Existing System

2.1.1 Purpose

This chapter reviews JWC's water system, documenting the age, condition, and capacities of each key process within the JWC WTP and components of the water transmission system. This effort was completed to establish a clear understanding of the existing system and its assets before identifying specific needs, deficiencies, and opportunities for optimization, and recommending capital improvement projects to ensure the successful continuation of JWC operations for the next 20 years.

The asset descriptions and capacities presented in this chapter are derived from discussions with JWC staff as well as the following reference documents:

- *JWC Capital Improvement Master Planning Services: 2009 Master Plan* (2009 Master Plan).
- *Joint Water Commission Water Service Agreement* (January 2014).
- *Water Treatment Plant Facility Plan* (2018 Facility Plan).
- *2020 Water Management and Conservation Plan*.
- *JWC 2020 Operations Manual*.
- Plans and drawings from the 2020 *Water Treatment Plant Expansion to 85 mgd.*

2.2 JWC Water Treatment Plant

As of 2020, the JWC WTP is the largest conventional treatment plant in Oregon. It was originally constructed in 1976 and has undergone several major upgrade projects, between 1995 and 1998, in 2005, and again between 2018 and 2020. Under the most recent expansion project, hydraulic bottlenecks were removed to increase the plant's maximum capacity from 75 mgd to 85 mgd.

Ownership of the JWC WTP is split between the four partners with ownership percentages calculated based on the amount of plant flow that each member requires. Table 2.1 presents the ownership percentages of the partners.

Table 2.1 JWC WTP Ownership by JWC Partners

JWC Partner	Ownership Percentage	Ownership Flow (mgd)
City of Hillsboro	49.12%	41.75
City of Beaverton	22.06%	18.75
TVWD	17.06%	14.5
City of Forest Grove	11.76%	10
Total	**100%**	**85**

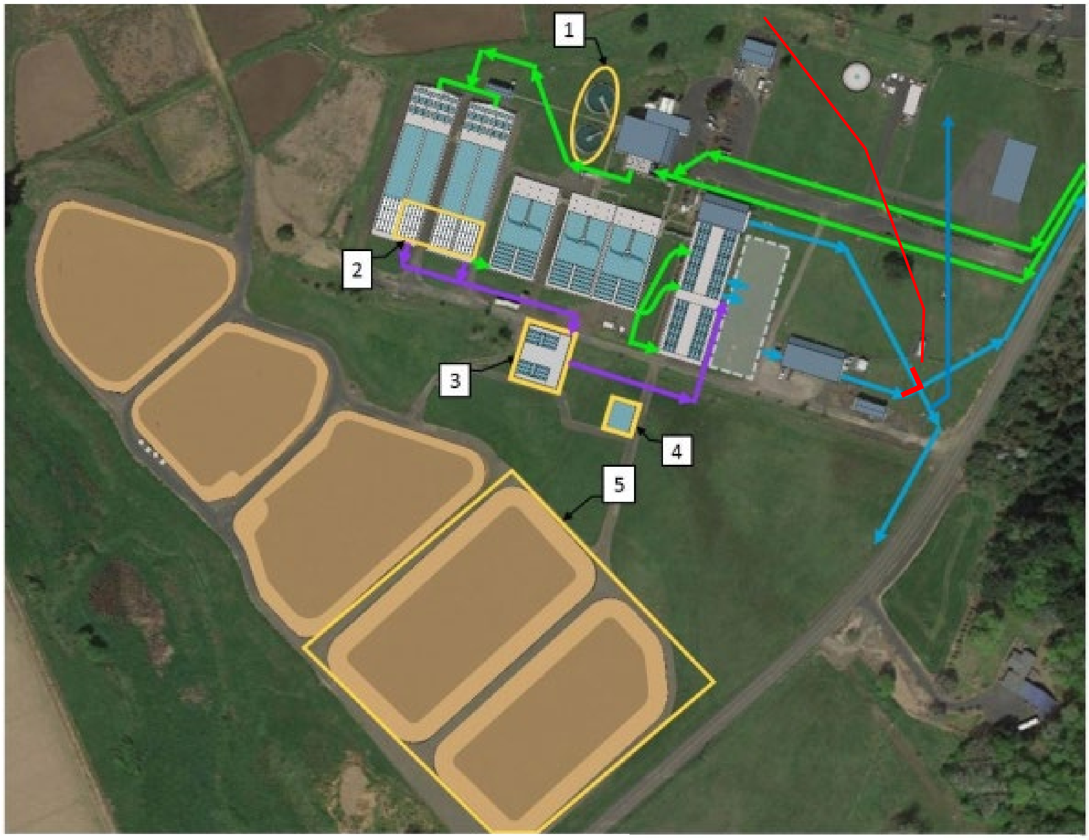
The facility is currently equipped with the following processes:

- Tualatin River intake.
- SHPP.
- Rapid mix basin.
- Flocculation and sedimentation (floc/sed) basins.
- Filtration and filter backwash.
- Clearwell.
- Finished water pump stations 1 and 2.
- Chemical systems, including the following components:
  - Aluminum sulfate (alum).
  - Coagulation/Settling Aid.

  - Chlorine gas.
  - Filter aid polymer.
  - Sodium hydroxide (caustic soda).
- Surge basins.
- Recycle pump stations.
- Solids drying beds.
- Decant pump stations 1 and 2.
- Underdrain pump station.
- Solids diversion pump station.
- Power distribution.
- Supervisory control and data acquisition (SCADA) and plant controls.
- Emergency generators.

The following sections discuss each of these processes and the current condition of their infrastructure and operations in further detail.

Figure 2.2 shows the JWC WTP's site plan as presented in the 2018 Facility Plan, amended with the 24-inch FG/Dilley transmission main, and Figure 2.3 shows the plant's process flow diagram as presented in the 2020 expansion drawings.



LEGEND

1. **Abandoned gravity thickeners**
2. **New plate settlers**
3. **New filters**
4. **New surge basin and recycle pump station**
5. **New sludge-drying beds**

LEGEND (cont'd)

🟩 **Existing raw water**
🟦 **Existing finished water**
🟪 **New pipelines**

Figure 2.2 JWC WTP Site Plan

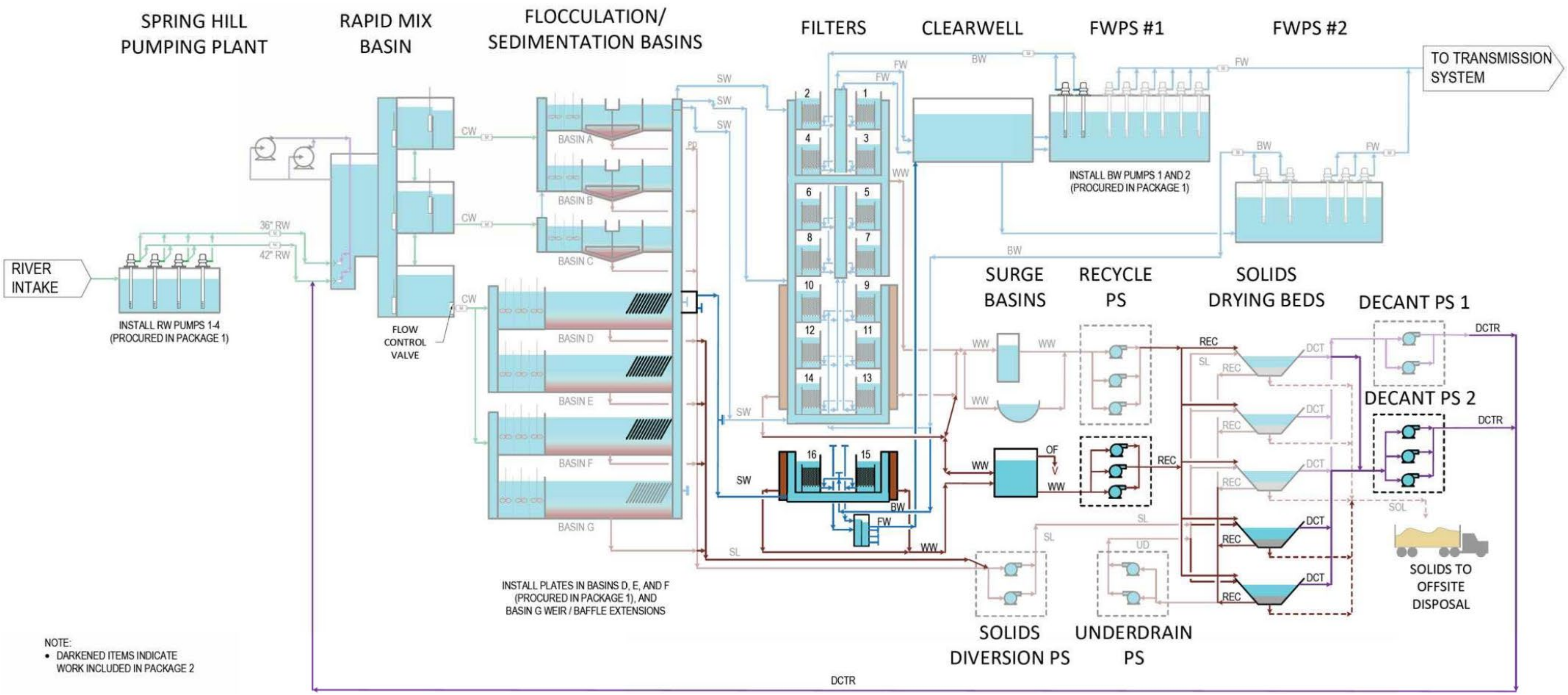


Figure 2.3 JWC Process Flow Diagram

2.2.1 Raw Water Impoundments

Raw water is impounded at two locations for use during the release season, when JWC is not on natural river flows, Barney Reservoir and Scoggins Reservoir, both of which have different storage capacities, with ownership percentages split between the four JWC partners and other agencies.

2.2.1.1 Barney Reservoir

Barney Reservoir, originally constructed in 1970, is fed from the Middle Fork of the Trask River's North Fork. The Trask River watershed covers an area of approximately 175 square miles, and the sub-watershed diverted to Barney Reservoir covers approximately 13.2 square miles.

The reservoir is formed by the Eldon S. Mills Dam, a 120-foot rock-fill dam with a total storage capacity of 20,000 acre-feet. When the reservoir is full, 14,886 acre-feet of the total capacity are available to JWC. Combined, the release rate for JWC and Clean Water Services (CWS) is limited to 68.7 cubic feet per second (cfs) (44.4 mgd), due to water rights and infrastructure capacity. JWC's portion is released from Barney Reservoir and diverted from the Trask River watershed to the Tualatin River watershed through the 36-inch pipeline that diverts water across the Coast Range to the headwaters of the Tualatin River. Water released from the reservoir becomes available for diversion at the SHPP intake in approximately 24 hours given the water's travel time from the reservoir to the intake, especially during seasons of low flow.

Barney Reservoir is managed by the Barney Reservoir Joint Ownership Commission, which is comprised of TVWD, CWS, Hillsboro, Forest Grove, and Beaverton, with Hillsboro serving as the managing agency and with joint ownership with the Joint Operating Committee.

Stored water is allocated to each member as well as the Oregon Department of Fish and Wildlife (ODFW), which is not an owner but is entitled to release 15 percent of stored water to the Trask River during low-flow seasons to support fish and wildlife. Furthermore, 2 percent of the total reservoir volume is inaccessible and referred to as a "dead pool." These additional factors explain why reservoir ownership and water allocation values do not align in Table 2.2. Table 2.2 describes reservoir ownership and water allocations.

Table 2.2 Barney Reservoir Storage per Agency

Agency	Reservoir Ownership (%)	Water Allocation (%)	Storage at Full Capacity (acre-feet)
TVWD	35%	29%	5,789
Hillsboro	31%	26%	5,127
Beaverton	21.5%	18%	3,556
Clean Water Services	10%	8%	1,654
Forest Grove	2.5%	2%	414
ODFW	0%	15%	3,000
Dead Pool	0%	2%	460
Total	**100%**	**100%**	**20,000**

2.2.1.2 Scoggins Reservoir

Scoggins Reservoir, also known as Henry Hagg Lake, was built in 1975 using a 150-foot earth dam between 1972 and 1975 for recreation, agriculture, water quality control, flood control, and municipal and industrial uses. The Scoggins Dam and Scoggins Reservoir are owned by the Bureau of Reclamation (USBOR) and operated by Tualatin Valley Irrigation District (TVID). The reservoir's total capacity is 56,000 acre-feet, of which approximately half is contracted to TVID.

JWC has up to 13,500 acre-feet in water rights when the reservoir is at full capacity. The USBOR has water rights contracts with Hillsboro, Forest Grove, and Beaverton for 5,000, 4,500, and 4,000 acre-feet of water, respectively. Water is released from Scoggins Reservoir to Scoggins Creek, a tributary of the Tualatin River. It takes approximately 12 hours for water released from the dam to reach the SHPP Intake under low flow conditions. Water is available to JWC from Scoggins Reservoir releases under two different water rights certificates: One of 13,000 acre-feet that has a release rate of 70 cfs and another owned by Hillsboro for 500 acre-feet, which does not have a release rate limitation.

Table 2.3 shows the storage capacity ownership volumes and active storage percentages are shown for Scoggins Reservoir for each of the JWC members.

Table 2.3 Scoggins Reservoir Storage per Agency

Agency	Active Storage at Full Capacity (acre-feet)	Active Storage (%)
Hillsboro	5,000	9.3%
Forest Grove	4,500	8.4%
Beaverton	4,000	7.5%
TVWD	0	0%
Clean Water Services	12,618	23.5%
TVID	27,022	50.4%
Lake Oswego Corporation	500	0.9%
Total	**53,640**	**100%**

2.2.2 SHPP Intake and Raw Water Pump Station

2.2.2.1 Overview

JWC's raw water intake and pump station are located at the SHPP on the Tualatin River. The SHPP intake is owned by the USBOR and shared between JWC and TVID. JWC has 4 pumps and TVID has 10 separate pumps inside the pump station, respectively. The total capacity is 88 mgd, with a firm capacity of 65 mgd.

JWC is responsible for maintaining smaller sections of the bar screens, which amount to approximately 1/3 of the total bar screen surface area. JWC is specifically responsible for the bar screens located in front of the travelling fish screens used for the JWC pumps on the intake, which are manually raked clean on a regular schedule. Meanwhile, TVID is responsible for maintenance and operation of the other 10 pumps and two screens as well as dredging the Tualatin River near the intake structure to keep it clear of sediment and debris.

Inside the SHPP, there are two Evoqua travelling fish screens that protect any fish caught in the intake for the raw water pumps and divert them back to the Tualatin River upstream of the intake screens. The travelling screens are routinely maintained and are scheduled to be evaluated during an upcoming condition assessment.

Raw water passes through the bar and travelling screens to the raw water pumps. JWC owns and operates four 400-horsepower (hp) vertical turbine pumps, each with a capacity of 15,500 gallons per minute (gpm) at 79 feet of total dynamic head and driven by variable frequency drives (VFDs). The pumps, which were replaced as part of the most recent expansion project, move water through two raw water pipelines, 36 and 42 inches in diameter, from the intake structure to the rapid mix basin at the WTP. Each of the raw water pipelines is equipped with an insertion flow meter upstream of the rapid mix basin. The plant recycle line enters the 42-inch raw water pipeline just downstream of the meter and upstream of the rapid mix basin.

Table 2.4 shows the design characteristics for the equipment inside the SHPP.

Table 2.4 Spring Hill Pumping Plant Design Characteristics

Equipment	Installation Year	Capacity	Drive Type
Bar Screens	2015	Steel bar grating: 1-3/4 inch x 3/16 inch Area: 51 feet 6-3/4 inch x 15 feet 10-1/2 inch	n/a
Traveling Fish Screens	1974, 1980	No. 1: 25 mgd No. 2: 0.939 – 1.877 feet per second	n/a
Pumps 1 and 2	2020	400 hp; 15,500 gpm; 22.3 mgd	VFD
Pumps 3 and 4	2020	400 hp; 15,500 gpm; 22.3 mgd	VFD

The SHPP building is fed with electricity from the backup power facility that serves as the main power distribution for the WTP, which is fed from two substations located in Yamhill and Cornelius, providing electric grid power redundancy. Additionally, JWC has its own standby electrical supply at the WTP via the onsite diesel standby power generators, which can also be used to supply power to the raw water pumps. The WTP lost utility power during the widespread wildfires in September 2020, but the WTP stayed online due to the emergency generators.

2.2.2.2 Age and Condition

This section details the age and condition of each asset in the order they were introduced in the previous section.

Several sections of the JWC-operated bar screens were replaced in 2015 and are in serviceable condition. During the fall season each year, TVID stops pumping at the SHPP and closes the covers on their sections of the screens. The current from the continued JWC pumping draws material to the JWC section of the screens, which causes an increase in the accumulation of debris on the bar screens. This increases the maintenance load and the frequency and scale of screen cleanings.

The two traveling fish screens are from 1974 and 1980. Plant staff have routinely maintained and replaced gear, chains, and screen sections on the existing traveling screens, but are increasingly concerned about the condition of the bearings inside the screens.

All four of the SHPP's pumps were replaced in 2019. Pumps 1 and 2 run on VFD drives installed in 2005, which are a combination of soft start and VFD, however, the soft start does not currently work and the pumps currently only operate on VFDs. As of June 2021, JWC became aware that Pumps 1 and 2 are experiencing critical frequency issues that cause vibration. JWC is working on a vibration mitigation concept of bracing the pumps at the base and mid-column. Pumps 1 and 2 were offline for maintenance due to vibration issues identified during operation.

Pumps 3 and 4 run on VFD drives that were installed in 2020. During start-up in 2021, under low-flow conditions, these pumps emitted noise likely caused by cavitation. In response to the cavitation-type noises, testing showed that the pumps were operating off the design curve. A control algorithm was added to limit the top speed of the pumps based on river level and the number of pumps running, to ensure that these pumps continued operating on their design curves and to limit potential cavitation. Pumps 3 and 4 are operational, and these issues do not appear to significantly impact operations.

Finally, the SHPP building, which was built in 1974, does not meet current seismic standards. As part of this master plan, JWC is considering several alternatives to acquire a seismically resilient intake and raw water pump station at different locations. The existing electrical system, lack of space for additional pumps, and seismic issues limits the SHPP's total pumping capacity to 88 mgd, which does not allow for future capacity expansions.

2.2.3 Rapid Mix

2.2.3.1 Overview

From the SHPP intake, two raw water pipelines feed raw water to the WTP's 85-mgd rapid mix basin. Chlorine and caustic soda are fed upstream on the rapid mix basins in the meter vaults. Alum, chlorine, and caustic soda are fed into the rapid mix basin, with caustic soda injected in the raw water pipes just upstream of rapid mix to achieve a longer reaction time; caustic soda is not routinely added at this location and is mainly used during the fall and winter months. Powdered activated carbon (PAC) and coagulant aid can also be added, if necessary, but these chemicals are not routinely used.

The rapid mix basin utilizes a weir-based flow-splitting system to direct water to seven floc/sed basins, labeled A through G, with open basin jet injection used for chemical mixing. Feed to each floc/sed basin is controlled by gates or valves at the rapid mix basin to feed or isolate the downstream processes. A total of three manually-operated weir gates service Basins A-C two weir gates feed Basins A and B and one weir gate feeds Basin C. An electrically-actuated butterfly valve is continually operated to control combined flow to Basins D through G. Flow to Basins D-G is controlled to maintain a flow rate less than the sum of the raw water and the recycle flow, which causes water to back up into the rapid mix basin and then overflow the weir gates for Basins A-C.

Flow is fed preferentially to Basins D through G since their plate settlers improve sedimentation performance beyond what the other basins can achieve; however, enough flow is diverted to basins A through C to keep fresh water flowing.

2.2.3.2 Age and Condition

The rapid mix basin was built as part of the plant's original construction in 1976 and then expanded in 1990. In 1998, a pumped jet-injection rapid mixer was added to the open basin.

Then, in 2018, 40-hp jet-injection pumps were installed, operating in a one-duty-one-standby configuration.

JWC's 2015 update to the Capital Improvement Program noted that the existing rapid mix facility warrants replacement due to its seismic vulnerabilities. The 2018 Facility Plan recommended removing the fiberglass baffle wall and weir gates for Basins D through G and relocating the injection point to increase the capacity of the rapid mix basin from 65 mgd to 88 mgd. These upgrades were completed during the 2020 WTP Expansion, increasing the capacity from 75 mgd to 85 mgd.

2.2.4 Flocculation and Sedimentation Basins

2.2.4.1 Overview

The WTP has seven floc/sed basins, labeled Basins A through G. Basins A and B were constructed as part of the original WTP in 1976 and each have a capacity of 8.33 mgd. Basin C was added in 1990 and has a capacity of 8.33 mgd. Basins D though G were constructed as part of the 1998 expansion and each has a capacity of 15 mgd. All floc/sed basins are gravity sedimentation basins and have vertical turbine flocculators.

The sedimentation basins for A through C are equipped with electric flocculators, with mechanical circular sludge rakes in one portion and hoseless circular sludge collectors under the effluent collection launders. The sedimentation basins for D through G are equipped with electric flocculators and chains, and flight sludge collectors, as well as plate settlers that were installed in Basins D to G. Basin G was the first basin to receive a plate settler in 2005. The settlers for Basins D to F were installed in 2018.

Table 2.5 summarizes the design characteristics for floc/sed basins A through G.

Table 2.5 Floc/sed Basins A to G Design Characteristics

Basin	Installation Date	Basin Capacity (mgd)	Loading Capacity (gpm/sf)	Equipment
A & B	1974	8.33	TBD	Electrically-actuated flocculators; Mechanical sludge rakes; Hoseless sludge collectors
C	1990	8.33	TBD	Electrically-actuated flocculators; Mechanical sludge rakes; Hoseless sludge collectors
D to G	1998	15	0.29 (summer); 0.19 (winter)	Plate settlers; Chain/Flight Sludge Collectors; Electrically-actuated flocculators

Note:
Abbreviation: gpm/sf – gallons per minute per square foot.

2.2.4.2 Age and Condition

The existing floc/sed basins have several documented issues, predominantly associated with the sedimentation process. Some of these issues have been addressed and resolved, as discussed below.

First, floc/sed Basins A through C are square basins equipped with circular mechanical rakes. The 2009 Master Plan noted that the difference in geometry between the square basin and circular sludge collectors causes sludge to build up in the corners of the basin, which cannot be reached by the collectors. Plant staff routinely removes this sludge using fire hoses to clean out the corners of the basin, but this process requires the sedimentation basin to be drained.

Second, as noted in the 2009 Master Plan, Basin G lacks flow control, which makes routing of its 15-mgd capacity through the unit difficult. However, recent adjustments to the sedimentation basins' weirs made the basins more hydraulically efficient. After these adjustments were made, flow to the basins is more consistent, even without the availability of more concise flow control.

Also noted in the 2009 Master Plan, a 6-inch drain line routes through the solids diversion pump station and connects to all of the basins. Though this line was designated to only serve Basins A - C, it has historically backed up its fluids and materials into the other basins, specifically when one basin is down for cleaning. The exact cause of this backup is unknown. Although plant staff installed an additional 12-inch pipeline during a recent expansion of the sludge collection system, this problem persists. Recent investigations by plant staff found clogging on the 6-inch line between Basins D - E and Basins F - G that contributed to the backup. Staff are also investigating changing the controls for the solids diversion pump station to minimize backup events.

Finally, the sedimentation basins' historical process performance had previously been suboptimal, specifically in regard to the plant's inability to meet the Oregon Health Authority's (OHA's) settled water turbidity recommendations during peak turbidity events. To improve performance, JWC installed plate settlers in Basins D through F when, previously, only Basin G was equipped with plate settlers. Plant staff confirmed that settled water turbidity has significantly improved since these installations, and no such issues have been reported since.

2.2.5 Filters

2.2.5.1 Overview

The filtration system is comprised of 16 dual-bay filters with a total combined surface area of 14,462 square feet (sf). Settled water flows out of sedimentation Basins A through G into Filters 1 through 14 via a combination of two 48-inch pipelines and a 60-inch pipeline that connect the influent channel to a splitter box on the east end of Basin A. Located at the end of Basin D, a 48-inch pipe feeds into the settled water channel for Filters 15 and 16.

Individual effluent pipelines carry filtered effluent out of Filters 1 through 8 and discharge directly into the clearwell whereas Filters 9 through 14 discharge into combined effluent pipes that carry filtered water to the clearwell underneath Filters 1 through 8. Filters 15 and 16 tie into the clearwell further downstream of the discharge location for Filters 1-14, which results in some loss of overall contact time (CT). This configuration is also discussed in terms of disinfection in Section 2.2.6.

Table 2.6 shows the design characteristics for filters 1 through 14, which have an approved peak loading capacity of 8.7 gpm/sf. The filter actuators on Filters 1 through 8 are a mix of electric, pneumatic, and hydraulic while those on Filters 9 through 16are all electric. A filter backwashing system, including both water and air scour components serves as the cleaning mechanism for all 16 filters, which are covered below in more detail in Sections 2.2.6 and 2.2.7. Figure 2.4, as shown in technical memorandum (TM) 2-1: Full Scale Filter Performance Demonstration (CH2M), illustrates the filter media configuration using 1.1-millimeter (mm) anthracite over 0.6-mm sand, which sits on top of a block-style underdrain.

Table 2.6 Filters 1 through 14 Design Characteristics

Parameter	Design Criteria
Number of Filters	14
Number of Bays per Filter	2
Bay Dimensions	231 sf
Hydraulic Loading Rate	8.7 gpm/sf
Filter Media Anthracite (1.1 mm) Depth	50 inches
Filter Media Sand (0.6 mm) Depth	6 inches
Filter Underdrain Type	Xylem-Leopold Type S HDPE Dual Lateral
Filter Influent Channel Capacity	105 mgd
Combined Filter Effluent Conduit Capacity	105 mgd
Air Scour Loading Rate	2 to 4 scfm/sf
Maximum Backwash Hydraulic Loading Rate	23 gpm/sf

Note:
Abbreviations: HDPE – high-density polyethylene; scfm – standard cubic feet per minute; scfm/sf – standard cubic feet per minute per square foot.

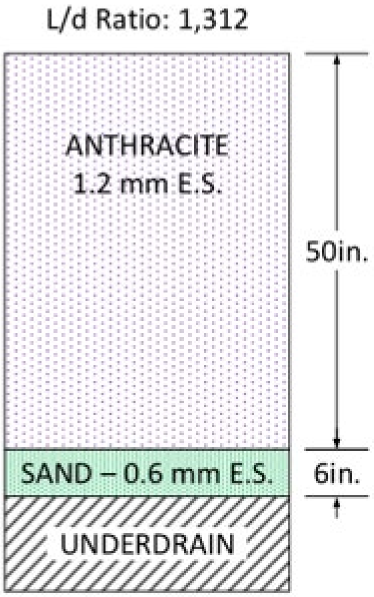


Figure 2.4 Media Configuration for Filters 1 through 14

As shown in the design characteristics presented in Table 2.7, Filters 15 and 16 have two 462-sf bays per filter, with an OHA-approved loading rate of 12 gpm/sf; however, a maximum hydraulic loading rate of 10 gpm/sf is used during normal operations. Similar to Filters 1-14, the media configuration of Filters 15 and 16 also uses anthracite and sand, but with different media depths and with the media sitting on top of stainless steel underdrain trilaterals. Figure 2.5, as shown in TM2-1: Full Scale Filter Performance Demonstration (CH2M), illustrates this filter configuration.

Table 2.7 Filter 15 and 16 Design Characteristics

Parameter	Design Criteria
Number of Filters	2
Number of Bays per Filter	2
Bay Dimensions	462 sf
Hydraulic Loading Rate	10 gpm/sf
Filter Media Anthracite Depth	72 inches
Filter Media Sand Depth	12 inches
Filter Underdrain Type	Stainless Steel
Filter Influent Channel Capacity	108 mgd
Combined Filter Effluent Conduit Capacity	108 mgd
Air Scour Loading Rate	2 to 4 scfm/sf
Maximum Backwash Hydraulic Loading Rate	23 gpm/sf

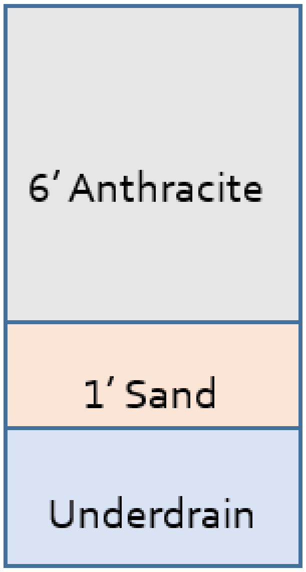


Figure 2.5 Media Configuration for Filters 15 and 16

2.2.5.2 Age and Condition

The 16 filters were all installed at different times: Filters 1 through 4 were built as part of the original 1976 construction, and Filters 5 through 6 were added during the 1984 expansion. Filters 7 through 8 were added during the 1991 expansion, and Filters 9 through 14 were added during the 1996 expansion. Filters 1-14 received new media in 2018. Most recently, Filters 15 and 16 were installed during the 2020 expansion.

The 2009 Master Plan noted that the mix of different valve actuator types (air-actuated, pneumatically-actuated, and electrically-actuated) that serve Filters 1 through 8 render maintenance difficult. This issue is ongoing.

Filters 15 and 16 provide a seismically sound foundation for the filtration facility, which will be expanded during the plant's ongoing seismic resiliency upgrades.

2.2.6 Filter Backwash

2.2.6.1 Overview

Vertical turbine Backwash Pumps 1 and 2 are located in Finished Water Pump Station (FWPS) 1, both with a capacity of 4,120 gpm, and service Filters 1 through 8. Meanwhile, vertical turbine Backwash Pumps 3 and 4, with a capacity of 9,200 gpm (each), are located in FWPS 2 and service Filters 9 through 16. Normally, only one filter is backwashed at a time, though two filters could be backwashed at the same time, under certain circumstances.

Plant staff operate the backwash pumps with significant flexibility and ease. The backwash control system prompts the operator to designate a time interval between backwashes. Then, the control system measures the level in the surge basin to calculate the required pumping rate. This pumping rate is set according to the water turbidity and fill rate of the equalization basins.

Table 2.8 shows the design characteristics for Backwash Pumps 1 through 4.

Table 2.8 Backwash Pumps Design Characteristics

Backwash Pump Number	Capacity per Pump (gpm)	Location	Filters Served
1 & 2	4,210	FWPS 1	Filters 1 through 8
3 & 4	9,200	FWPS 2	Filters 9 through 16

2.2.6.2 Age and Condition

Filter Backwash Pumps 1 and 2 were replaced in 2018 and operate on duty/standby mode. Backwash Pumps 3 and 4 were installed during the 1998 expansion and operate on duty/standby mode. JWC staff plan to upgrade Backwash Pumps 3 and 4 from soft starts to VFDs in Summer 2021.

2.2.7 Air Scour System

2.2.7.1 Overview

Two air scour blowers located in FWPS 2 service the WTP's 16 filters. Each is rated for 985 standard cubic feet per minute (scfm) and have a combined flow rate of 1,970 scfm. Table 2.9 shows the design characteristics for the air scour blowers.

Table 2.9 Air Scour Blower Design Characteristics

Parameter	Value
Blower Type	Positive Displacement
Number of Blowers	2
Power (each)	60 hp
Capacity (each)	985 cfm
Air Scour Rate	4.0 cfm/sf
Pressure	8 psig

Note:
Abbreviations: cfm – cubic feet per minute; cfm/sf - cubic feet per minute per square foot; psig – pounds per square inch gauge.

2.2.7.2 Age and Condition

The air scour blowers were installed during the 1998 expansion. As of June 2021, JWC staff have experienced issues with random tripping of the drives. To fix these issues, JWC staff plans to replace the 1998 soft starts with new soft starts in 2021.

2.2.8 Disinfection

Filtered water effluent discharges into the clearwell where the CT for disinfection begins. The covered clearwell is a baffled, reinforced concrete basin with a design capacity of 1.5 million gallons (MG).

As can be seen in Figure 2.6, which is taken from the 2018 Facility Plan, the clearwell has two inlets, one for effluent from Filter 1 through 14 and the other for effluent from Filters 15 and 16. There are two outlets that discharge to the FWPS, with one discharging to FWPS 1, located at the end of the clearwell, and the second discharging to FWPS 2, located upstream of FWPS 1's discharge. The clearwell has a truncated flow path due to the locations of the tie-ins for effluent from Filters 15 and 16 and the outlet to FWPS 2; JWC staff originally wanted to add this new effluent tie-in to the Filters 1-14 effluent tie-in, but was deemed by the contractor to be too complex and expensive, thus leading to the selection of the current location of the Filter 15-16 effluent tie-in. This truncated flow path decreases CT, thereby reducing the clearwell capacity to 1.2 MG, as opposed to its design capacity of 1.5 MG. This is a relatively new issue that became apparent when Filters 15 and 16 were added in 2020 and must continue to be monitored. Figure 2.6 has been modified from its original version to show the approximate location of the inlet for filtered water from Filters 15 and 16.

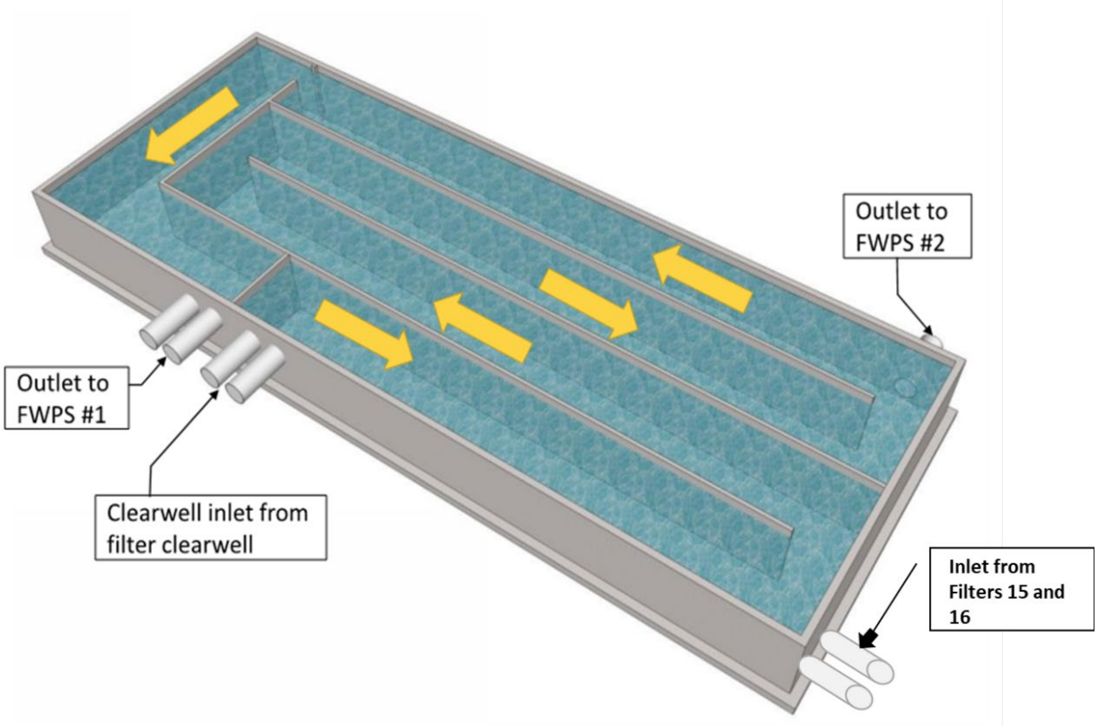


Figure 2.6 Clearwell Configuration

Treated water is disinfected using chlorine gas, which can be fed to four process locations:

- Raw water at the rapid mix basin.
- Settled water to Filters 1 through 14 and to Filters 15 and 16.
- Combined filter effluent from Filters 1 through 14 and Filters 15 and 16.
- Finished water upstream of FWPS 1 and 2.

There are baffles in the clearwell upstream of the discharges to FWPS 1 and 2, which aid in mixing the finished water with the chlorine. CT is partially met in the clearwell, with the remainder achieved in the distribution pipelines. The WTP is the first user and CT is achieved in the pipeline feeding the WTP operations (OPS) building. At the WTP OPS building, flow and water quality values are monitored for CT calculations. CT for the rest of the distribution system is achieved in the distribution pipelines that discharge to the finished water reservoirs.

2.2.8.1 Age and Condition

The 2018 Facility Plan noted that the clearwell was previously evaluated and deemed seismically deficient. In addition, the evaluation also determined that the existing structure's vulnerabilities cannot be mitigated to meet current seismic standards and requires replacement. Furthermore, the volume of the existing clearwell is also deficient due to the installation of the new tie-ins for effluent from Filters 15 and 16 and the discharge tie-in for FWPS 2. The following concerns related to clearwell volume were noted:

- Achieving adequate CT in the existing clearwell is challenging, especially since new tie-ins for effluent from Filters 15 and 16 and the outlet to FWPS 2 were installed. These new inlets and outlets essentially reduce the total available clearwell volume from 1.5 MG to approximately 1.0 MG.

- The current volume limits filter backwashing operations, as each backwash can use approximately one hundred thousand gallons, equivalent to nearly 10 percent of overall clearwell volume.
- The typical rule of thumb for clearwell volume is 10 percent of peak treated water production. While this rule is not absolute, the 1.5 mgd clearwell (operationally reduced to only 1.0 MG) is significantly below this rule (~2 percent of peak WTP production).

Due to these factors, JWC is considering an expansion of the clearwell. The 2018 Water Treatment Plant Facility Plan recommended replacing the existing clearwell with a new 2.5 MG clearwell.

Currently, the JWC WTP is the first user of finished water, via a pipe running back to the plant off the finished water transmission mains. This configuration allows additional CT to be taken from residence time in the transmission mains. It is of note, to meet CT requirements per OHA, a pump from FWPS 1 must be in service at all times.

2.2.9 Finished Water Pumping and Distribution

2.2.9.1 Overview

Finished water leaves the clearwell at two points and is distributed to the transmission system from FWPS 1 and 2. FWPS 1 feeds into a 42-inch transmission line and FWPS 2 feeds into 72-inch transmission line. Chlorine is added upstream of each pump station to provide a trim dose for the chlorine residual throughout the distribution system.

FWPS 1 has a nameplate capacity of 46 mgd and houses six vertical turbine pumps; meanwhile, FWPS 2 has a nameplate capacity of 39 MG and houses three vertical turbine pumps with two open bays for additional pumps in the future. Table 2.10 shows the design capacity of all nine pumps.

Table 2.10 Finished Water Pump Stations Design Criteria

Pump Number	Pump Station Location	Pump Capacity (gpm / mgd)	hp
1	FWPS 1	4,000 / 5.76	400
2	FWPS 1	7,000 / 10.1	800
3	FWPS 1	4,000 / 5.76	400
4	FWPS 1	4,000 / 5.76	400
5	FWPS 1	6,000 / 8.64	700
6	FWPS 1	7,000 / 10.1	800
7	FWPS 2	9,000 / 12.96	1,000
8	FWPS 2	9,000 / 12.96	1,000
9	FWPS 2	9,000 / 12.96	1,000

2.2.9.2 Age and Condition

Installed in 1976, FWPS 1 is not seismically resilient and is planned to be replaced with a future FWPS 3. The installation years for all pumps in FWPS 1 and 2 are shown below in Table 2.11, with many having undergone replacements, whether wholly or compartmentally, at various times between 1992 and 2015. Additionally, the pumps and motors for Pumps 1, 2 and 6 and the motor in Pumps 4are slated for near-term replacement.

The motor control center (MCC) that serves FWPS 1 was replaced in 2016. Additional planned improvements include replacing the pump pedestals of those pumps being replaced and adding motor health monitoring.

FWPS 2 was installed during the 1998 expansion project, and its MCC was replaced in 2020.

Table 2.11 Finished Water Pump Installation Year

Pump Number	Installation/Replacement Year
1	2002
2	1992
3	2015
4	2013
5	2015
6	1992
7	1998
8	1998
9	1998

2.3 Chemical Systems

The JWC WTP uses multiple chemical systems to aid in coagulation, pH adjustment, sedimentation, filtration, and disinfection, consisting of alum, chlorine gas, coagulation aid, filter aid polymer, PAC, and sodium hydroxide (caustic soda). The following sections detail each system.

2.3.1 Aluminum Sulfate (Alum) System

2.3.1.1 Overview

Alum, the primary coagulant, is added at rapid mix to coagulate suspended solids and dissolved organic carbon in the raw water. The alum system is comprised of three storage tanks and metering pumps. Tanks 1, 2, and 3 have storage capacities of 13,900 gallons, 9,200 gallons, and 4,700 gallons, respectively. Additionally, three pumps, two duty and one standby, are dedicated to the 36-inch and 42-inch raw water pipes coming into the system.

Currently, the plant is limited in its alum storage capacity, which is a 13-day supply at 50-mgd flow and a 25-milligram-per-liter (mg/L) dose rate. The typical recommended design capacity for chemical systems is to provide is a 30-day supply at average flow and average dose. Table 2.12 shows the alum tank design characteristics.

Table 2.12 Alum Storage Tank Design Characteristics

Alum Storage Tank Number	Capacity (gallons)	Installation Year
1	13,900	1976
2	9,200	0/1
3	4,700	0/1

2.3.1.2 Age and Condition

Tank 1 was installed in 1976 and was blasted and relined in 2018. Tanks 2 and 3 were installed in 2000/1 and were recently inspected, but have not required maintenance. Maintenance regarding the metering pumps was not discussed with plant staff.

2.3.2 Chlorine Gas System

2.3.2.1 Overview

Chlorine gas is used for disinfection and a chlorine solution is fed into the system at seven locations. Approximately 3 tons per week are used for water treatment. Nine 1-ton cylinders of chlorine gas are kept in a room adjacent to the larger chemical room that houses alum, PAC, coagulant aid, and filter polymer aid. At any time, six cylinders are connected, with three cylinders feeding into the system and the remaining three cylinders on standby. The plant also has seven chlorinators, all of which operate under flow-paced control.

Table 2.13 presents capacity, dose location, and installation year of the chlorinators.

Table 2.13 Chlorinator Design Characteristics

Unit	Capacity (ppd)	Application Location	Installation Year
1	1,000	Settled Water to Filters 1 through 14	TBD
2	1,000	Filters 1 through 14 Combined Filter Effluent	TBD
3	150	FWPS 2	TBD
4	150	FWPS 1	TBD
5	750	Rapid Mix	TBD
6	250	Settled Water to Filters 15 and 16	2020
7	500	Filters 15 and 16 Combined Filter Effluent	2020

Note:
Abbreviation: ppd – pounds per day.

A key issue with the chlorine gas system is that the system is limited to only one supplier, who has only two sources in the region. Given past issues with this limited supply, plant staff stated that they prefer to employ a bulk sodium-hypochlorite system or on-site hypochlorite generation system instead of the existing chlorine gas system.

2.3.2.2 Age and Condition

The chlorine gas system was installed during various projects, with some piping being dated to 1996. The chlorine storage tanks are owned and switched out by the vendor, so JWC is not responsible for their maintenance. The vacuum piping is 16 years old, and JWC plans to replace it in 2021. The chlorine scrubber is serviced every four years. Two chlorinators were added in 2005, and two new chlorinators were installed during the 2018 expansion. Replacement of the chlorine gas system with hypochlorite is a priority for JWC staff. The expansion of the chlorine gas system was meant as an interim upgrade until the switch to hypochlorite is made.

2.3.3 Coagulation/Settling Aid Systems

2.3.3.1 Overview

The coagulation aid/settling aid systems were installed in 2005 and upgraded in 2020. The coagulation aid/ settling aid system can be used to enhance coagulation but is rarely used because the floc/sed basin has produced settled water of acceptable turbidity without the use of coagulation aid. Settling aid to drying beds has not yet been used since the system's installation. The system consists of two liquid polymer blending units, operating on duty/standby, and a 10 gallon per hour (gph) neat polymer pump.

The settling aid system can also be used to add polymer to enhance settling in the solids drying beds. This system consists of a polymer blending unit skid, 0.95 gph neat polymer pump, 100 gallon day tank, and two polymer pumps with a maximum feed rate of 35.19 gph (flow rate dependent on tube size and roller installed). The pump set to feed polymer to the recycle water has tubes installed that provide a maximum capacity of 28.5 gph. The pump set to feed polymer to the solids diversion pump station has tubes installed that provide a maximum capacity of 10.1 gph. Both pumps are duty only with no standby pumps.

Both systems are set up to supply polymer from 275 or 330-gallon liquid polymer totes. Each system can be operated using the same polymer or separate polymers. There is storage for a maximum of two polymer totes. Currently both systems are set up to use the same polymer. The design characteristics for the coagulation/settling aid systems is shown below in Table 2.14.

Table 2.14 Coagulation/Settling Aid System Design Characteristics

Application Location	Feed Rate (gph)	Installation Year
Rapid Mix Basin	10	2005
Recycle Pump Station	28.5	2020
Solids Diversion Pump Station	10.1	2020

2.3.3.2 Age and Condition

The settling aid polymer system was installed between 2019 and 2020. No functionality issues or bottlenecks have been reported in this process.

2.3.4 Filter Aid Polymer System

2.3.4.1 Overview

The filter aid polymer system is used to enhance filter performance. A dry polymer batching system is used to create 320-gallon batches of dilute polymer solution (approximately 0.45 percent). Two sets of pumps feed to the filters, with both sets operating with two pumps in a duty-standby configuration. One set of pumps feeds to Filters 1 through 14 while the second set feeds to Filters 15 and 16. All pumps are capable of feeding a maximum of 158.5 gph of polymer solution, with the flow rate dependent on the tube size and roller installed in each metering pump. Currently, the tubes installed on pumps supplying Filters 1 through 14 are capable of feeding 44.4 gph, while the tubes installed on the pumps supplying Filters 15 and 16 can feed 28.5 gph. Table 2.15 shows the design characteristics for the Filter Aid Polymer System.

Table 2.15 Filter Aid Polymer System Design Characteristics

Application Location	Feed Rate (gph)	Pump Installation Year
Filters 1 through 14	44.4	2020
Filters 15 and 16	28.5	2020

2.3.4.2 Age and Condition

A new polymer mixing system and tank were installed in 2010. All four pumps were installed in 2020. No functionality issues or bottlenecks have been reported in this process.

2.3.5 Powdered Activated Carbon (PAC) System

2.3.5.1 Overview

The mgd PAC system is comprised of two feeders, bag storage, and a metering pump. PAC can be injected at the settled water flume or rapid mix basin. Each feeder system has an auger capable of feeding 275 pounds per hour (lb/hr) of PAC. The PAC system is only used for periodic taste and odor control events and algal toxin events; however, this system has not been used in 10 years since no qualifying events have occurred in that time.

2.3.5.2 Age and Condition

The first feeder and two pads were installed in 2008 after the previous carbon feed system was deemed to be insufficient in handling taste and odor events, with a second feeder installed in 2018 and a third feeder procured and slated for installment in 2021. However, since installing the new system, the JWC WTP has not experienced any recurring breakthrough events.

The PAC super sack feed system is located outside between the rapid mix basin and the floc/sed basins. JWC plans to move all chemicals to a new, seismically resilient chemical storage building sized to accommodate a new PAC feeder and limited quantities of super sack storage.

2.3.6 Sodium Hydroxide (Caustic Soda) System

2.3.6.1 Overview

Caustic soda is used to control the alkalinity and pH of the treated water. Four metering pumps add caustic soda at two points: to raw water vaults upstream of the rapid mix basin to control pH during extreme turbidity events or periods of low raw water alkalinity and to the clearwell to control the pH of the finished water.

This system has two seasonal operational modes, one for the summer months of May through October and one for the winter months of November through April. In the summer months, caustic soda is only dosed to the clearwell for pH adjustment, with the four metering pumps operating in a one-duty-three-standby configuration. In the winter months, caustic soda is dosed to the raw water upstream of the rapid mix basin, as needed, while maintaining constant dosing to the clearwell. In these instances, two metering pumps feed each location, operating on a one-duty-one-standby configuration. However, the raw water is dosed with caustic soda only when the alkalinity or pH drops. Alkalinity is more likely to drop than pH during big storm events.

All four pumps can provide a maximum feed rate of 158.5 gph, with the feed rate dependent on the tube size and roller installed. The two pumps dedicated to feeding the clearwell currently have a maximum capacity of 100 gph with the installed tubes. The two pumps that are used to feed either the clearwell or raw water have different tubes installed for summer and winter operation, with capacities of 100 gph and 50.7 gph respectively. Table 2.16 shows the design characteristics for the caustic soda system.

Table 2.16 Caustic Soda System Design Characteristics

Application Location	Number of Pumps	Feed Rate (gph)	Installation Year
Clearwell	1 duty + 1 standby	100	2018
Raw water/Clearwell	1 duty + 1 standby	Summer: 100; Winter: 50.7	2018

2.3.6.2 Age and Condition

The caustic soda system was installed in 2000, and a new chemical skid was added in 2018. The 2018 Facility Plan noted that, at that time, the system was unable to feed caustic soda at multiple locations and, as a result, the WTP experienced operational issues during a December 2015 storm event. This problem has been resolved.

JWC inspected the caustic soda system within the last three years and found no operational or maintenance issues. However, they plan to incorporate this system into the new chemical storage building, which will help consolidate chemical storage to a single area.

2.4 Solids Handling

The JWC WTP's solids are handled using surge basins, recycle pump stations, solids drying beds, decant pump stations, underdrain pump stations, and one solids diversion pump station. Figure 2.7 shows the process flow diagram of the solids handling processes. Each process is described in the following sections.

2.4.1 Surge Basins

2.4.1.1 Overview

The surge basins are the first process in the solids handling train. There are three surge basins that receive waste streams from filter backwash wastewater and the filter-to-waste water for Filters 1 through 16.

Basins 1 and 2 receive filter backwash water from Filters 1 through 8 and any overflows from Filters 9 through 16. Basins 1 and 2 have connected levels and both float on each other, matching the hydraulics for these two basins. Basin 3 receives flow from Filters 9 through 16 and overflows to Sedimentation Basin 2. Water is pumped from the surge basins via two recycle pump stations to the solids drying beds. Recycle Pump Station 1 services Basins 1 and 2 and Recycle Pump Station 2 services Basin 3.

Operation of the backwashes and the surge basins is set by JWC staff and was discussed in Section 2.2.6.1.

2.4.1.2 Age and Condition

Surge Basin 1 was constructed in 1974 as part of the original plant. Surge Basin 2 was added during the 1998 expansion while Surge Basin 3 was added during the 2018 expansion.

The design characteristics of the three basins is shown below in Table 2.17.

Table 2.17 Surge Basin Design Characteristics

Surge Basin Number	Installation Year	Capacity (gallons)	Depth at Capacity (feet)	Filters Served
Basin 1	1974	115,000	16	Filters 1 through 8
Basin 2	1998	125,000	15	Filters 1 through 8
Basin 3	2018	256,000	16	Filters 9 through 16

The 2009 Master Plan noted previous issues with Surge Basins 1 and 2 and the thickeners unable to handle all of the solids received from the floc/sed basins. However, with the addition of Surge Basin 3, the surge basins now have the necessary capacity to handle the plant's solids load; however, during storm events, JWC staff is still concerned about storage capacity due to the size and flowrates of Filter 15 and 16. Though the infrastructure is still standing, the thickeners were taken out of service upon completion of Surge Basin 3 and are planned to be demolished in the future.

2.4.2 Recycle Pump Stations

2.4.2.1 Overview

Water is pumped from the surge basins via two recycle pump stations to the solids drying beds. Recycle Pump Station 1 has three VFD pumps rated at 750 gpm to service Basins 1 and 2. Recycle Pump Station 2 uses three VFD pumps rated at 1,000 gpm each to service Basin 3.

From the pump stations, backwash wastewater is transferred to the solids drying beds.

Table 2.18 shows the design characteristics for the Recycle Pump Stations.

Table 2.18 Recycle Pump Station Design Characteristics

Recycle Pump Station	Number of Pumps	Pump Capacity (gpm)	Installation Year
Number 1	3	750	2005
Number 2	3	1,000	2018

2.4.2.2 Age and Condition

Recycle Pump Station 1 was restored with all new pumps in 2005 while Recycle Pump Station 2 was newly installed during the 2018 expansion. The system was previously hydraulically limited to a capacity of 1,300 gpm with two pumps operating and 1,800 gpm with three pumps operating by a small diameter pipe downstream of Recycle Pump Station 1. As part of the 2018 expansion, the discharge from Recycle Pump Station 1 was modified and now combines with the discharge from Recycle Pump Station 2 to the drying beds. Maximum flow rates have not substantially increased after the modifications. However, control strategies have changed with the addition of Recycle Pump Station 2 and Surge Basin 3, which has mitigated these limitations.

2.4.3 Solids Drying Beds

2.4.3.1 Overview

Five solids drying beds receive wastewater and solids from the filters, via the surge basins and recycle pump stations, and the floc/sed basins, via the Solids Diversion Pump Station. The capacity of beds 1 through 3 is 240,000 cubic feet (cf) at a depth of 4 feet. Beds 4 and 5 have a combined capacity of 326,000 cf at a depth of 3.5 feet. The maximum loading of the beds is 17 pounds per square foot (lb/sf).

Under normal operations, a bed is filled for three to six months before being rotated out, upon which solids are sent to a different bed. This rotation of the beds allows for small layers to be created in each bed, which generates a more efficient drying system. Typically, the beds dry the solids to between 40 and 60 percent solids, after which the solids are disposed of off-site via truck hauling.

2.4.3.2 Age and Condition

Solids drying beds 1 through 3 were installed in 2005 and only had central laterals; then more laterals were installed in solids drying beds 1, 2, and 3 in 2016, 2014, and 2013, respectively, to improve drainage to the underdrain system. Beds 4 and 5 were installed as part of the 2020 expansion.

The design characteristics for the Solids Drying Beds are shown below in Table 2.19.

Table 2.19 Solids Drying Beds Design Characteristics

Solids Drying Bed Number	Capacity (cf)	Depth (feet)	Max Loading Rate (lb/sf)	Installation Year
1	240,000	4	17	2005
2	240,000	4	17	2005
3	240,000	4	17	2005
4 & 5	326,000 (combined)	3.5	17	2018

2.4.4 Decant Pump Stations

2.4.4.1 Overview

From the solids drying bed, two decant pump stations transfer water to the front of the plant, feeding into the 42-inch raw water pipeline just before the rapid mix basin. Decant Pump Station 1 has two submersible 10-hp pumps each with a capacity of approximately 520 gpm while Decant Pump Station 2 has three submersible pumps, each with an 1,800-gpm capacity at 35 feet of head that are run with VFDs. Table 2.20 shows the design characteristics for the Decant Pump Stations.

Table 2.20 Decant Pump Stations Design Characteristics

Decant Pump Station	Number of Pumps	Pump Capacity (gpm)	Installation Year
Number 1	2	520	2005
Number 2	3	1,800	2018

2.4.4.2 Age and Condition

Decant Pump Station 1 was constructed in 2005. Decant Pump Station 2 was installed during the 2018 expansion.

While the pumps in Decant Pump Station 1 are each 520 gpm, the combined pumping capacity is limited. When the second pump is run concurrently, there is only a minimal increase of 50 to 100 gpm in flow. However, these pumps are rarely used after the addition of Decant Pump Station 2 and this hydraulic limitation does not result in any operational limitations. No issues exist with the newly installed Decant Pump Station 2.

2.4.5 Underdrain Pump Station

2.4.5.1 Overview

The underdrain pump station is used for two scenarios: it is used to move water from the underdrains of the solids drying beds back up to the top of the beds or when JWC staff drain a basin and encounter dirty water, which is then pumped to an online drying bed. There are two pumps with a capacity of approximately 100 gpm. Table 2.21 shows the design characteristics for the Underdrain Pump Station.

Table 2.21 Underdrain Pump Station Design Characteristics

Number of Pumps	Pump Capacity (gpm)	Installation Year
2	100	2005

2.4.5.2 Age and Condition

The Underdrain Pump Station was installed in 2005. No functionality issues exist for this system.

2.4.6 Solids Diversion Pump Station

2.4.6.1 Overview

The Solids Diversion Pump Station transfers solids from the floc/sed basins to the solids drying beds and has two 15-hp pumps, each with a capacity of approximately 400 gpm, that operate on a duty/standby configuration. Table 2.22 shows the design characteristics for the Solids Diversion Pump Station.

Table 2.22 Solids Diversion Pump Station Design Characteristics

Number of Pumps	Pump Capacity (gpm)	Installation Year
2	400	2005

2.4.6.2 Age and Condition

The Solids Diversion Pump Station was installed in 2005.

Capacity limitations in the Solids Diversion Pump Station and hydraulics in the piping network also limit the capacity of the settled solids system. To mitigate these limitations and impacts, JWC staff are currently implementing control strategy changes.

2.5 SCADA System

2.5.1 Overview

The SCADA system provides plant control to operations staff. All major operational systems are incorporated into the SCADA system.

2.5.2 Age and Condition

There have been three recent major upgrades to the SCADA system. The first upgrade occurred in 2005, when the overall full system was changed to PLC. The next update occurred in 2016, when the SCADA software was updated to Wonderware. The last SCADA update occurred in 2018 during the WTP expansion, during which time the surge basin pumping programming was optimized.

2.6 Standby Power Feed

2.6.1 Overview

The standby power feed system is comprised of two 25 megawatt diesel generators, a generator control panel/fuse box, and a day tank for fuel storage. A total of 15,000 gallons of fuel storage is available but the tanks are filled only to 90 percent capacity, providing approximately 9,500 to 13,700 gallons of fuel storage. The standby power generator is sized to provide 37.5 mgd of finished water from the WTP.

2.6.2 Age and Condition

The standby power system was installed in 2015. While the rated capacity is 37.5 mgd of finished water, the system was able to provide 50 mgd during an outage that lasted for 42 hours.

2.7 Priority Future WTP Improvements

Due to budget constraints, some of the following improvements were not completed as part of the 85-mgd WTP expansion completed in 2020: Other projects listed include top priority improvements that were not in the scope of the 2020 WTP expansion:

- Addition of more alum storage through installation of additional alum tanks:
  - JWC plans to incorporate this element into a new chemical storage building, with PAC and chlorine being top priority for rehousing.
  - The new chemical building may be built in phases.
- Replacement of chlorine gas with sodium hypochlorite.
- Demolition of solids thickeners.
- FWPS 1 pump upgrades.

2.8 Transmission System

Finished water from the JWC WTP is distributed to JWC agencies via the NTL, STL, and FG lines, and is stored in the Fern Hill reservoirs. Finished water from FWPS 1 and 2 directly serve JWC customers via the NTL and FG lines, with excess water sent to the Fern Hill reservoirs. From the reservoirs, water is served by gravity to JWC partners and wholesale customers, primarily via the STL. Figure 2.7 illustrates the system's hydraulic profile.

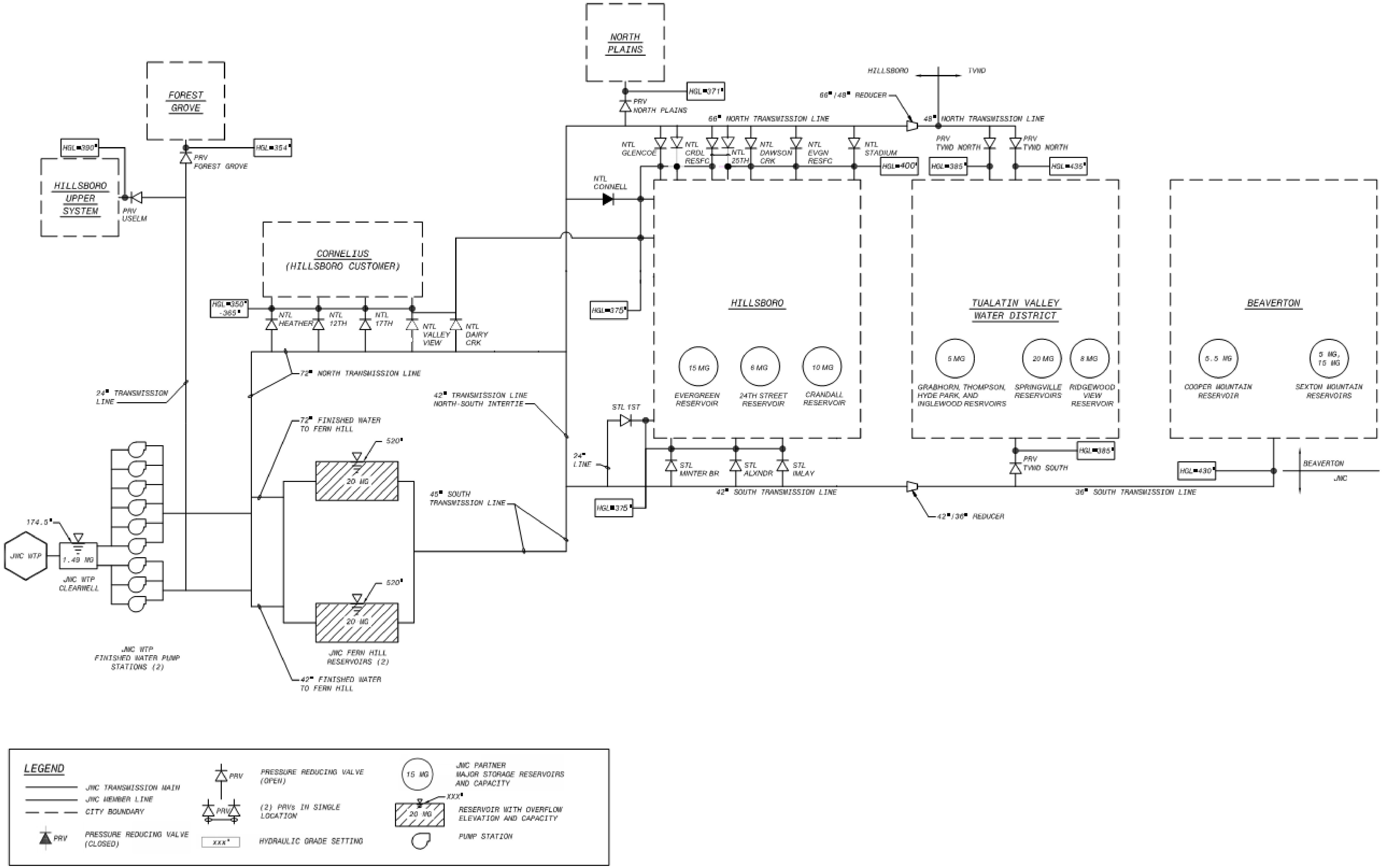


Figure 2.7 Hydraulic Profile of JWC System

2.8.1 Fern Hill Reservoirs

2.8.1.1 Overview

The Fern Hill Reservoirs are comprised of two 20-MG prestressed concrete reservoirs. Two transmission mains, a 72-inch-diameter pipeline and a 42-inch-diameter pipeline, transfer finished water from the WTP to a mixing valve box where the water from the two pipelines is mixed. Finished water leaves the valve box and fills the two reservoirs. Finished water is distributed from the reservoirs to JWC's partners primarily through the STL.

Ownership of the reservoirs is split between the JWC members, whose ownership percentages and volumes are shown in Table 2.23.

Table 2.23 Fern Hill Reservoir Ownership by JWC Partners

JWC Partner	Percent Ownership	Ownership Volume (MG)
City of Hillsboro	45%	10.8
City of Beaverton	25%	6
TVWD	16.67%	4
City of Forest Grove	13.33%	3.2
Total	**100%**	**24**

Reservoir 1 was constructed early in the 1980s, and Reservoir 2 was constructed in 2005. Onsite facilities include an electrical and water quality monitoring building which is located downhill of the reservoirs. This building houses chemical testing equipment, which is used for both inline monitoring of pH, chlorine, turbidity, conductivity, and temperature, as well as for analyzing grab samples taken by the operators. The reservoirs are not used to meet the required CT for disinfection. If more CT is needed beyond the clearwell, the transmission mains are used.

JWC incorporates several considerations into the reservoirs' operations. JWC must consider changing demands for each partner's daily flow, plus incorporate the travel time required to release and distribute flows from the reservoirs. In order to retain the operational flexibility necessary to balance these considerations, these assets are operated within an operational band of 20-percent capacity, or 8 MG, which translates to the top 7 feet of the reservoirs. Another 20 percent, or 8 MG, is reserved for emergencies and the remaining 60 percent, or 24 MG, is split between JWC members' ownership, as shown in Table 2.23.

Design characteristics for reservoirs 1 and 2 are shown below in Table 2.24.

Table 2.24 Finished Water Reservoirs Design Characteristics

Reservoir Number	Storage Volume (MG)	Diameter (feet)	Height (feet)	Bottom Elevation (feet)	Operational Range (feet)	Installation Year
1	20	350	30	491	23 to 30	1980s
2	20	350	30	490	23 to 30	2005

A seasonal operations consideration during the autumn involves reservoir cycling after the first significant autumnal storm, which is needed to accommodate the increased chlorine residual in the reservoir due to increase chlorine demand, which is necessary to treat increased flows of water to the WTP seen from these storm events.

In addition to seasonal fluctuations, JWC must accommodate fluctuations in partner demands. Swings in demand are caused by the numerous partner connection points to the JWC transmission mains and how the pressure-reducing valves (PRVs) and flow rates are controlled. Due to this dynamic, concurrent fill and drawdown events can induce large swings in the JWC system.

Given these circumstances, JWC staff would like to move from the current pressure-flow control to a combination of flow and pressure control valves. This transition is currently underway, with staff from both JWC and Hillsboro working together to establish dual-flow control mode.

2.8.2 Transmission Mains

Water from the Fern Hill reservoirs is distributed to JWC members through the STL, the NTL, and a 24-inch FG transmission main. The installation date of these transmission mains varies between 1974 to the late 2000's.

2.8.2.1 South Transmission Line

Shown in Figure 2.1, the STL was installed between 1974 and 1980 and is comprised of 45- and 42-inch-diameter bar-wrapped concrete cylinder pipes. This pipeline delivers water from the reservoirs to the southside of Hillsboro and south connection of TVWD and ends at the Beaverton connection.

More specifically, the STL runs as a 45-inch-diameter pipe east on Blooming Fern Hill Road and Tongue Lane until it intersects Highway 219, along which it heads north until it meets the NTL-STL 42-inch transmission main connection on the south end of Jackson Bottom Wetland. From there, the STL reduces to a 42-inch-diameter pipe and heads east until it reaches the intersection of SW Minter Bridge Road before heading north and continues along the south side of the Tualatin Valley Highway and then east until it reaches proximity with SW Cornelius Pass Road where it ends at the 36-inch connection with Beaverton (the 36-inch line belongs to Beaverton, not to JWC)

Ownership and capacity of the STL is split between TVWD, Hillsboro, and Beaverton whose exact percentages are shown in Table 2.25.

Table 2.25 STL Ownership by JWC Partners (please divide this on 45-inch and 42-inch STL section)

JWC Partner	Percent Ownership	Flow Ownership (mgd)
City of Hillsboro	52.6%	20
City of Beaverton	36.9%	14
TVWD	10.5%	4

2.8.2.2 North Transmission Line

Also shown in Figure 2.1, the NTL was completed in 2002 and consists of 66- and 72-inch-diameter mortar-lined and coated steel piping. This pipeline also includes the 7,728-foot-long, 42-inch intertie that joins the NTL and STL in southwest Hillsboro. The NTL provides water to Cornelius and the north side of Hillsboro and ends at the TVWD north connection near the intersection of Hwy 26 and Cornelius Pass Road.

More specifically, the NTL begins as a 72-inch-diameter pipeline that conveys water north on Fern Hill Road and continues west of CWS's Forest Grove Wastewater Treatment Plant. Then, it heads east and crosses over both public and private land as it heads to the intersection of Heather Street and Mountain View Lane in the south end of Forest Grove before continuing east on Heather Street and passing into Cornelius through 10th Avenue, Beech Street, and 12th Avenue until it meets the Tualatin Valley Highway. From there, the NTL turns east to enter Hillsboro, where it meets the NTL-STL intertie main at the intersection of SE Dennis Avenue and Baseline Street. The pipeline then turns north along Dennis Avenue, Main Street, NW Cornell Road, and NW Glencoe Road until it meets Evergreen Road, at which point it turns east and travels along Evergreen Road, where it changes diameters from 72 to 66 inches. The 66 inch-diameter NTL continues east to just after the connection to Crandall reservoir, upon which it turns northeast and terminates at the connection point of the TVWD pipeline at NW Cornelius Pass Road.

Like the STL, ownership and 80 mgd capacity of the NTL is split between TVWD, Hillsboro, and Beaverton whose exact percentages are shown in Table 2.26.

Table 2.26 NTL Ownership by JWC Partners

JWC Partner	Percent Ownership	Flow Ownership (mgd)
City of Hillsboro	48.6%	38.95
City of Beaverton	2.63%	2.1
TVWD	48.6%	38.95
Total	**100%**	**80**

2.8.2.3 24-Inch Transmission Main

Also shown in Figure 2.1, the 24-inch-diameter concrete cylinder pipe transmission main services Forest Grove and Hillsboro’s Upper System, which was built in 1977. This main branches off the 42- and 72-inch-diameter transmission lines at the plant, that carry water from FWPS 1 and 2 to the Fern Hill reservoirs.

As shown in Table 2.27, ownership of the 24-inch transmission main is split evenly between Forest Grove and Hillsboro.

Table 2.27 24-inch Diameter Transmission Ownership by JWC Partners

JWC Partner	Percentage Ownership
City of Hillsboro	50%
City of Forest Grove	50%

2.8.2.4 Member Agency Turnouts

PRVs are used to control flow from the three transmission mains out to the JWC members. Table 2.28 lists all the turnout locations on the STL, NTL, and 24-inch transmission main.

Table 2.28 Member Agency Turnout Locations on 45-inch/42-inch-diameter STL

Turnout Location	PRV Size (inch)	Meter Size (inch)	Customer Served
S 1st Ave and Maple	8, 12	8	Hillsboro
TV Hwy and Minter Bridge	4, 8	4	Hillsboro
TV Hwy and Imlay (south)	10	10	Hillsboro
TV Hwy and Imlay (north)	10	10	Hillsboro
TV Hwy and SE 75th Ave	4	12	TVWD
TV Hwy and Cornelius Pass	None	36	Beaverton
Jackson Bottom	None	1.5	Hillsboro
Clean Water Services	2	2	Hillsboro
TV Hwy and Cornelius Pass	None	30	Hillsboro (temporarily)

Note:
Abbreviations: TV – Tualatin Valley; Hwy – highway.

Table 2.29 Member Agency Turnout Locations on 72-inch/66-inch-diameter NTL

Turnout Location	PRV Size (inch)	Meter Size (inch)	Customer Served
10th and Heather meter 1	8	8	Cornelius
10th and Heather meter 2	2	2	Cornelius
12th and Baseline meter 1	6	6	Cornelius
12th and Baseline meter 2	3	3	Cornelius
17th and Baseline meter 1	6	6	Cornelius
17th and Baseline meter 2	3	3	Cornelius
Valley View on TV Hwy meter 1	3	3	Hillsboro
Valley View on TV Hwy meter 2	6	6	Hillsboro
TV Hwy west of Dairy Creek	6	3	Hillsboro
Connell and Jackson	3, 10	3	Hillsboro
Glencoe and Evergreen meter 1	3, 6	12	Hillsboro
Glencoe and Evergreen meter 2	10	10	North Plains
25th and Evergreen meter 1	6	18	Hillsboro
25th and Evergreen meter 2	10	18	Hillsboro
Evergreen Reservoir	10	12	Hillsboro
Shute and Evergreen meter 1	6, 12	18	Hillsboro
229th and Bennett meter 1	6	18	Hillsboro
229th and Bennett meter 2	12	18	Hillsboro
Cornelius Pass and Hwy 26	24	24	TVWD

Table 2.30 Member Agency Turnout Locations on 24-inch FG/Dilley Line

Turnout Location	PRV Size (inch)	Meter Size (inch)	Customer Served
Elm St and Hwy 47 Bypass meter 1	8	8	Hillsboro
Elm St and Hwy 47 Bypass meter 2	2	2	Hillsboro
Elm St and Hwy 47 Bypass meter 3	8, 24	24	Forest Grove

2.8.3 Pipeline Condition Assessment

Available in Appendix E, the *Pipeline Condition Assessment Framework* TM summarizes prior inspection work performed on the STL and NTL using data and recommendations from reports written in 2020, 2009, and 2008. The TM also provides a reference table of available assessment technologies, including their relative costs, operational effects during testing, and applicable pipe materials for each technology.

Prior condition assessment work was centered around the pipelines' age, construction practices employed during their construction, possible corrosion, and internal and external pressure changes experienced during operations. The last comprehensive condition assessment was completed during the development of the 2009 Master Plan and the implementation of certain recommendations made for the STL and NTL are still pending.

These recommendations, which are detailed in the following sections, remain pertinent and are noted as recommended actions in this present master plan. In addition to pipeline-specific actions, JWC is recommended to take coupon samples of pipes that have failed or require modification and to complete coupon evaluations.

Finally, a more complete evaluation of the transmission system was conducted as part of this present master plan using a transient pressure evaluation of JWC's hydraulic models. The results of this evaluation are described in Chapter 7.

2.8.3.1 STL Recommendations

JWC is recommended to install piezoelectric hydrophones into high-risk areas of the STL to monitor leakage. These high-risk areas can be identified using soil survey data, among other available data, and are further recommended to be evaluated using remote field eddy current/transformer coupling (RFEC/TC) or pulsed eddy current (PEC). The information gathered using these inspection technologies can be utilized to perform a nonlinear finite element analysis and determine the pipe conditions that may lead to leaks and ruptures.

Additionally, any locations identified as experiencing joint mortar degradation are recommended to be reinspected. Other areas not previously identified as experiencing degradation should still be inspected using non-destructive methods to determine the extent of the mortar coating corrosion.

2.8.3.2 NTL Recommendations

According to the 2009 Master Plan, the following monitoring activities were recommended for the NTL within the next 10 years:

- Conduct soil-resistivity testing and analyses to determine soil characteristics along the length of the pipeline. The results can be used to project concrete-coating deterioration rates, estimate pipe life, and identify locations of likely corrosion. As of October 2021, this has been partially completed and JWC plans to continue in the future.
- Conduct ultrasonic testing to estimate wall thickness and areas experiencing pitting corrosion. This work requires excavating the pipe and removing its coating. Future monitoring locations can be determined using leak history and other contributing factors. As of October 2021, this has not yet started.
- Install piezoelectric hydrophones into high-risk areas to monitor for leaks. As of October 2021, this has not yet started.

Chapter 3

PLANNING CONSIDERATIONS

3.1 Water System Policies and Planning Criteria

The Joint Water Commission (JWC) manages their water system according to policies and criteria that are organized into the following categories:

- Ownership guidelines.
- Planning criteria.
- Production strategy.
- Sustainability goals.
- Maintenance policies.
- Design standards.

This section details each of these categories to establish a firm understanding of JWC's existing policies and operational protocols. In addition, identified planning criteria will be used throughout this Water System Master Plan (Plan) to evaluate the existing system. The policy framework established in this section also sets expectations for what this Plan's recommended operational and capital improvements must meet to support JWC in meeting their service goals.

3.1.1 Ownership Guidelines

This section describes existing ownership guidelines and policies that apply to JWC's water system. Ownership of system components is shared amongst the JWC partners at agreed upon percentages. The partners also select the Managing Agency that will operate and maintain the system. At the time of this Plan, the City of Hillsboro is the Managing Agency.

3.1.1.1 Barney Reservoir

The Barney Reservoir is owned by the Barney Reservoir Joint Ownership Commission, which consists of the four JWC partner agencies and Clean Water Services (CWS). Table 3.1 summarizes ownership percentages and reservoir capacities of the Barney Reservoir, as established in the *Joint Ownership Agreement Barney Project*, which is also included in Exhibit A of the *JWC Water Service Agreement*.

Table 3.1 Barney Reservoir Ownership

Agency	Reservoir Percent Ownership (%)(1)	Reservoir Storage Ownership at Full Capacity (ac-ft)(1)(2)
Hillsboro	31.0	6,200
Beaverton	21.5	4,300
TVWD	35.0	7,000
Forest Grove	2.5	500
CWS	10.0	2,000
Total	**100**	**20,000**

Notes:
Abbreviations: ac-ft – acre-feet.
(1) Actual amount of water available is dependent on the reservoir level, fish flow requirements, dead storage, and inflow.
(2) Ownership percentage and capacity established in Exhibit A of the JWC Water Service Agreement.

While reservoir storage ownership is divided amongst the members of the Barney Reservoir Joint Ownership Commission, available storage volumes for use by each JWC partner are based on separate water allocation percentages. Note that costs related to the Barney Reservoir remain shared amongst the members of the Barney Reservoir Joint Ownership Commission based on ownership percentages, not water allocation percentages. The water allocation percentages account for the fact that a portion of the stored water in the reservoir is allocated to additional uses, thus limiting the volume of stored water each partner may utilize. For one, as shown in Table 3.2 below, 15 percent of the stored water in Barney Reservoir is allocated to the Oregon Department of Fish and Wildlife (ODFW). An additional two percent is classified as dead pool storage and is unavailable for use. As a result, the total volume of stored water each JWC partner may utilize is lower than their ownership capacity.

Table 3.2 summarizes each agency's water allocation percentages and storage capacities available to each partner or agency at full capacity for the Barney Reservoir.

Table 3.2 Barney Reservoir Water Allocation

Agency	Water Allocation (%)(1)	Reservoir Storage Available at Full Capacity (ac-ft)(1)(2)
Hillsboro	26	5,127
Beaverton	18	3,556
TVWD	29	5,789
Forest Grove	2	414
CWS	8	1,654
ODFW	15	3,000
Dead Pool	2	460
Total	**100**	**20,000**

Notes:
(1) Actual amount of water available is dependent on the reservoir level and inflow.
(2) Water allocation percentage and available storage at full capacity taken from 2020 Operations Manual.

3.1.1.2 Hagg Lake and Scoggins Dam

Scoggins Dam and Hagg Lake were built by the Bureau of Reclamation (USBOR) as part of the Tualatin Reclamation Project, which was authorized by Congress in 1966. JWC's partners have the rights to utilize a portion of the stored water in Hagg Lake for municipal uses. In addition to the JWC, CWS, Tualatin Valley Irrigation District (TVID), and the Lake Oswego Corporation hold water rights to stored water in Hagg Lake.

Original agreements with the USBOR describing water rights allocations are provided in Exhibit O of the *JWC Water Service Agreement*, with the water rights also summarized in Exhibit B. Table 3.3 summarizes the stored water rights in Hagg Lake for each partner or agency.

Table 3.3 Hagg Lake Storage Ownership Allocation

Agency	Reservoir Storage Available at Full Capacity (ac-ft)(1)
Hillsboro	5,000
Beaverton	4,000
TVWD	0
Forest Grove	4,500
Clean Water Services	12,618
TVID	27,022
Lake Oswego Corporation	500
JWC Partner Total	**13,500**
Total Storage	**53,640**

Note:
(1) Stored water rights detailed in Exhibit B of the JWC Water Service Agreement.

3.1.1.3 Water Treatment Plant Facilities

Ownership of the JWC water treatment plant (WTP) and its ancillary facilities and infrastructure, including the raw water pipelines, is divided amongst the partners according to the plant's recently expanded peak capacity of 85 million gallons per day (mgd). Table 3.4 summarizes ownership percentages and capacities for the WTP facilities, as established in Exhibit F of the *JWC Water Service Agreement*.

Table 3.4 Raw Water Pipelines and WTP Ownership

Partner	Percent Ownership (%)(1)	Capacity (mgd)(1)
Hillsboro	49.12	41.75
Beaverton	22.06	18.75
TVWD	17.06	14.50
Forest Grove	11.76	10.00
Total	**100**	**85.00**

Note:
(1) Ownership percentage and capacity established in Exhibit F of the JWC Water Service Agreement.

Constructed in 2016, the standby power-generation facility was sized to operate the WTP at 37.5 mgd, 50 percent of the plant's 75-mgd capacity at the time of its commissioning. Ownership of the standby power-generation facility is divided amongst the partners according to this 37.5-mgd capacity, as described in Exhibit E of the *JWC Water Service Agreement* and summarized in Table 3.5. Ownership rates and capacities have remained unchanged by the plant's expansion to 85 mgd.

Table 3.5 Standby Power Generation Facility Ownership

Partner	Percent Ownership (%)(1)	Capacity (mgd)(1)
Hillsboro	67.62	25.36
Beaverton	12.40	4.65
TVWD	16.67	6.25
Forest Grove	3.31	1.24
Total	**100**	**37.5**

Note:
(1) Ownership percentage and capacity established in Exhibit E of the JWC Water Service Agreement.

3.1.1.4 Transmission Lines

Ownership of the 42-inch and 72-inch finished water transmission lines to the Fern Hill Reservoirs and the Fern Hill Reservoirs is explained in Exhibit E of the *JWC Water Service Agreement* and summarized in Table 3.6. Note that the ownership percentages for the finished water transmission lines and the Fern Hill Reservoirs are the same; however, the capacities shown apply only to the reservoirs.

Table 3.6 Finished Water Transmission Lines and Fern Hill Reservoirs Ownership

Partner	Percent Ownership (%)(1)	Capacity (MG)(1)(2)
Hillsboro	45.00	18.00
Beaverton	25.00	10.00
TVWD	16.67	6.67
Forest Grove	13.33	5.33
Total	**100**	**40.00**

Notes:
Abbreviation: MG – million gallons.
(1) Ownership percentage and capacity established in Exhibit E of the JWC Water Service Agreement.
(2) Capacity applies only to the Fern Hill Reservoir Storage volume.

Ownership of the transmission lines serving the partners, the North Transmission Line, South Transmission Line, and the 24-inch Transmission Line from the JWC WTP to Forest Grove, is shared amongst the individual partners served by each line. Ownership percentages are established in Exhibit D and capacity information is detailed in Exhibit K of the *JWC Water Service Agreement*.

Table 3.7, Table 3.8, and Table 3.9 break down ownership of the North Transmission Line, South Transmission Line, and the 24-inch Transmission Line from the JWC WTP to Forest Grove, respectively.

Table 3.7 North Transmission Line Facilities Ownership

Partner	Percent Ownership (%)(1)	Capacity (mgd)(2)
Hillsboro	48.685	38.95
Beaverton	2.63	2.10
TVWD	48.685	38.95
Forest Grove	0	0
Total	**100**	**80.00**

Notes:
(1) Ownership percentage established in Exhibit D of the JWC Water Service Agreement.
(2) Transmission line capacity and methodology for determination of capacity detailed in Exhibit K of the JWC Water Service Agreement.

Table 3.8 South Transmission Line Ownership

Partner	Percent Ownership (%)(1)	Capacity (mgd)(2)
Hillsboro	52.60	20.00
Beaverton	36.90	14.00
TVWD	10.50	4.00
Forest Grove	0	0
Total	**100**	**38.00**

Notes:
(1) Ownership percentage established in Exhibit D of the JWC Water Service Agreement.
(2) Transmission line capacity and methodology for determination of capacity detailed in Exhibit K of the JWC Water Service Agreement.

Table 3.9 Ownership of the 24-inch Transmission Line from the JWC WTP to Forest Grove

Partner	Percent Ownership (%)(1)	Capacity (mgd)(2)
Hillsboro	50	7.10
Beaverton	0	0
TVWD	0	0
Forest Grove	50	7.10
Total	**100**	**14.20**

Notes:
(1) Ownership percentage established in Exhibit D of the JWC Water Service Agreement.
(2) Transmission line capacity and methodology for determination of capacity detailed in Exhibit K of the JWC Water Service Agreement.

3.1.1.5 Billing Meters

As established in the *2020 Operations Manual*, JWC's policy is to own, operate, and maintain their system through, and including, each of the billing meters located at the turnouts of each partner agency and wholesale customers.

In the past, each partner owned and maintained their own billing meters, and ownership transfer of the billing meter from the partners to JWC is still ongoing. All meters previously owned by the City of Hillsboro have now been transferred to JWC ownership. The City of Forest Grove's meters were also replaced and transferred to JWC ownership. A strategy for installing new JWC-owned meters near the City of Beaverton and TVWD turnouts is under development.

3.1.2 Planning Criteria

This section describes planning criteria related to operating the JWC water system, including the Fern Hill Reservoirs' storage policies, partner systems' storage protocols, and minimum hydraulic grade lines (HGL).

3.1.2.1 Storage

JWC operates the Fern Hill Reservoirs to maximize partner storage, meet the WTP's operating requirements, and respond to emergencies. Storage in the Fern Hill Reservoirs is divided into three components, each with specific volumes of water:

- **Operational storage**: 20 percent.
- **Partner storage**: 60 percent.
- **Emergency storage**: 20 percent.

The normal minimum operating level is set to maintain a reservoir storage capacity of 60 percent.

Table 3.10 summarizes JWC's storage policies related to management of the Fern Hill Reservoirs.

Table 3.10 Fern Hill Reservoir Storage Policies

Policy	Source
If reservoir levels were to drop below the normal minimum reservoir storage (60 percent) due to circumstances other than a specific partner requesting to use their storage rights, then each partner's ownership of the remaining stored water will be reduced by its Fern Hill Reservoir ownership percentage.	2020 Operations Manual
If operations lead to reservoir levels that the Managing Agency determines to be critical, storage ownership rates will not apply, and remaining storage will be allocated for JWC operational requirements.	2020 Operations Manual
JWC partners may request use of stored water in addition to normal treatment capacity ownership under normal conditions.	2020 Operations Manual

In addition to storage at the Fern Hill Reservoirs, the *2020 Operations Manual* includes a policy that each partner must maintain and operate their own storage facilities and/or be able to utilize an alternative water source capable of meeting three days of average day demand. Capacity from aquifer storage and recovery facilities and regularly maintained wells can be used to meet this requirement, provided that the pumping capacity has a backup power supply.

3.1.2.2 Minimum Hydraulic Grade Line

JWC supplies water at the necessary HGL to operate the pressure reducing valves (PRVs) at each partner interconnection. JWC has two minimum HGL requirements, one for the end of the North Transmission Line and one for the end of the South Transmission Line, summarized in Table 3.11. Provided JWC meets the minimum HGL at these locations, the HGL at all other partner interconnections will be sufficient to operate all other PRVs.

Table 3.11 JWC Partner Minimum HGL Requirements

Location	Minimum HGL (feet)(1)
End of North Transmission Line – TVWD interconnection	450
End of South Transmission Line – End of Cornelius Pass 42-inch-diameter pipeline	430

Note:
(1) Minimum HGL elevation based on City of Hillsboro's 2014 revised benchmark elevation of 194.22. JWC transmission main design was originally based on NGVD29 Washington County Benchmark elevation of 194.31.

3.1.2.3 Maximum Transmission Main Velocity

Transmission main velocity shall not exceed 10 feet per second.

3.1.3 Production Strategy

This section describes the overall operational strategies and policies that guide the JWC system's operation as well as the production of water for the partner agencies. JWC operates the WTP and water system for 24 hours per day, 7 days a week, and 365 days a year with the goal of meeting each partner's system demand while maintaining adequate levels within the Fern Hill Reservoirs, which fluctuate in response to water demands and as needed to maintain water quality goals.

The following sections detail specific policies related to operational changes made by partners, seasonal operations, peak season water use reporting, curtailment, and service interruptions.

3.1.3.1 Partner Operation Changes

If a partner must make changes to system operations, they must give notice to the WTP in advance (as defined in the *2020 Operations Manual*) of any planned actions. In the event a partner has an unplanned flow disruption or requires additional flow, the partner must immediately notify the WTP.

3.1.3.2 Peak and Non-Peak Season Operations

The WTP's current water supply source is primarily from the Tualatin River and its tributaries and is supplemented, once the JWC partner water rights are regulated off, with stored water from Hagg Lake/Scoggins Dam and Barney Reservoir/Mills Dam. JWC's water year is divided into non-peak and peak seasons: During the non-peak (i.e., non-release) season, partner demands are met using JWC partners natural flow water rights in the Tualatin River while, during the peak (i.e., release) season, available natural flow in the Tualatin River is supplemented by storage releases from Hagg Lake and Barney Reservoir.

Table 3.12 summarizes key policies related to the WTP's water production in peak and non-peak seasons.

Table 3.12 Peak and Non-Peak Season Operation Policies

Policy	Source
During the non-peak (i.e., non-release) season, partners may request flow changes at any time and as often as required.	2020 Operations Manual
During peak (i.e., release) season, JWC requires partners to submit finished water requests each day to determine stored water release amounts.	2020 Operations Manual

3.1.3.3 Peak Season Water Use Accounting and Reporting

Table 3.13 summarizes JWC's key policies related to accounting and reporting water usage during the peak (i.e., release) season.

Table 3.13 Peak Season Water Use Accounting and Reporting

Policy	Source
During periods when natural flow water rights are regulated by Oregon Water Resources Department, the daily finished water demands of the owners of any remaining unregulated natural flow water rights will be satisfied first from those unregulated water rights before those owners will be deemed to have placed a demand on stored water.	2020 Operations Manual
The Managing Agency shall maintain and distribute records of stored water allocations, including total water available, both from natural flow and from stored water releases and quantities of stored water used by each partner, and the remaining net balance of owned and leased stored water.	2020 Operations Manual
The Managing Agency will distribute reports of finished water delivered to each partner and wholesale customers and remaining balance of owned and leased stored water for each Partner.	2020 Operations Manual
JWC reports daily release amounts for each contract holding partner to TVID every month.	N/A

Note:
Abbreviation: N/A – not applicable.

3.1.3.4 Curtailment

The JWC maintains a curtailment plan in accordance with Oregon Administrative Rules (OAR) Chapter 690 Division 86. Table 3.14 summarizes JWC's policies related to curtailment.

Table 3.14 Curtailment Policies

Policy	Source
JWC Commissioners have adopted a curtailment plan outlining protocols for emergency water shortages. The curtailment plan is updated on a regular basis, approximately every five years.	2020 Operations Manual, JWC Curtailment Plan
The curtailment plan will be implemented according to the criteria outlined in the curtailment plan and decision tree.	2020 Operations Manual, JWC Curtailment Plan
Curtailment decisions related to WTP production issues not addressed by the curtailment plan and decision tree will be implemented when recommended by the JWC WTP Manager and approved by the JWC General Manager.	2020 Operations Manual, JWC Curtailment Plan
Each JWC partner and wholesale customer is required to either have a curtailment plan or be willing to follow an applicable JWC partner curtailment plan.	2020 Operations Manual, JWC Curtailment Plan

3.1.3.5 Service Interruptions

Table 3.15 lists JWC's existing policies related to service interruptions.

Table 3.15 Service Interruption Policies

Policy	Source
The Managing Agency, at least 72 hours prior to planned service interruptions, shall notify each Partner.	2020 Operations Manual
A partner may object to planned service interruptions for operational reasons. The Managing Agency shall make reasonable efforts to reschedule interruptions to accommodate the objection.	2020 Operations Manual
Planned service interruptions or shutdowns during release season are scheduled such that they will not impact partner demands.	2020 Operations Manual
Partners will be notified as soon as possible in the event of an emergency service interruption.	2020 Operations Manual

3.1.4 Sustainability Goals

The JWC Events and Education Committee was formed in 2001 to coordinate each partner agency's respective conservation efforts. JWC oversees public participation, joint public messaging, and outreach efforts in conservation and water science.

All JWC partners are active members in the Regional Water Providers Consortium, an organization that promotes water conservation, source water protection, and the value of water. In addition, each partner maintains their own robust conservation programs, which are detailed in the *2020 Water Management and Conservation Plan.*

3.1.4.1 Recommended Sustainability and Conservation Policies

JWC may consider adopting these policies in the future:

- JWC shall update the JWC's Water Management and Conservation Plan every five years and the JWC partners will continue to promote conservation with their customers through their programs.
- JWC will continue to implement JWC's Source Water Protection Program and take necessary steps to ensure that water supply sources are protected for future community needs.
- JWC is environmentally sustainable and will seek opportunities to implement sustainable programs as feasible, including participating in regional and national water utility climate associations such as the Water Utility Climate Alliance.
- JWC will investigate energy conservation opportunities and alternative energy sources and generation.

3.1.5 Maintenance and Security Policies

Table 3.16 summarizes JWC's existing policies related to asset and information security.

Table 3.16 Security Policies

Policy	Source
Site visits or tours of the facilities by persons other than JWC staff are restricted. Authorization for site visits must be pre-approved. Requests for authorization must be directed to the JWC Managing Agency.	2020 Operations Manual
Access to records related to JWC facilities by persons other than JWC staff is restricted. Authorization for access to these records by other persons, including guests, consultants, suppliers, and vendors, must be through the Managing Agency's public records request procedure and may require a non-disclosure agreement.	2020 Operations Manual
The JWC SCADA system is isolated from external access.	2020 Operations Manual

Note:
Abbreviation: SCADA – supervisory control and data acquisition.

3.1.5.1 Recommended Maintenance Policies

JWC should consider adopting the following maintenance policies in the future:

- JWC will perform condition assessments of critical assets. JWC will develop and implement a condition assessment schedule for all critical assets.
- JWC will conduct maintenance activities at a level that is consistent with optimizing system reliability, asset economic life, and system performance.
- JWC will maintain and train highly qualified personnel that meet require operator certifications. Operators will satisfy all continuing education requirements. Note, this is an ongoing JWC policy.
- JWC will inspect and clean reservoirs every 1 to 5 years.
- JWC will perform leak detection analysis on 25 percent of the system each year.
- JWC will test all billing meters at interconnections yearly.

3.1.5.2 Additional Recommended Standards

JWC is recommended to develop additional design standards and policies related to resiliency, reliability, and redundancy, including seismic design standards and policies for required spare equipment and materials or retainer agreements with key suppliers in the event of an emergency. JWC should consider using Willamette Water Supply System's (WWSS's) design standards as the baseline in developing their own standards.

3.2 Resilience Level of Service Goals

A resilient water system is one that can rapidly recover after a major natural disaster to supply water for the critical needs of customers. This section summarizes the level of service (LOS) that JWC aims to provide to the partners as well as the performance objectives for water system infrastructure that would be required to achieve these LOS goals following an earthquake with a 72-year return period, a magnitude 9.0 (M9.0) Cascadia Subduction Zone (CSZ) earthquake, and an earthquake with a 2,475-year return period. Table 3.17 shows the return periods and probability of exceedance of each of these earthquakes.

JWC's 2008 Seismic Hazard Mitigation Study identifies three broad seismogenic sources for the general area of the JWC WTP:

- A mega-thrust source at an interface between the North American and Juan de Fuca plates in the CSZ.
- A deep subcrustal zone (intra-slab) in the subducted Juan de Fuca Plate and Gorda plates in the CSZ.
- A shallow crustal zone within the forearc of North American Plate.

While shallow crustal zone earthquakes generally result in 20 to 60 seconds of shaking, according to the Oregon Department of Emergency Management (OEM), the M9.0 CSZ earthquake that has the potential to hit Oregon will consist of an estimated 2 to 4 minutes of shaking or rolling.

The ground motion hazard at a given probability of exceedance is the sum of the hazard from the various seismogenic sources, which are calculated by the United States Geological Survey (USGS) using probabilistic seismic hazard analysis (PSHA). Detailed information on the nature of each earthquake event discussed in this chapter can be found in JWC's 2008 Seismic Hazard Mitigation Study.

Table 3.17 Earthquake Return Period and Probability of Exceedance

Earthquake Event	Return Period (years)	Probability of Exceedance in 50 Years
72-year	72 years	50 percent
CSZ	475 years	10 percent
2,475-year	2,475	2 percent

JWC is recommended to follow guidelines for water systems outlined in the 2013 *Oregon Resilience Plan* (ORP), a state-wide document that sets goals for phased recovery in the initial days and weeks following an M9.0 CSZ earthquake to support rapid economic and social recovery (OSSPAC, 2013)[1]. Many Oregonians expect water service to be restored within one

---

[1] *The Oregon Resilience Plan, Reducing Risk and Improving Recovery for the Next Cascadia Earthquake and Tsunami*. Oregon Seismic Safety Policy Advisory Commission. Salem, OR.

month after a major earthquake (City Club, 2017)[2], which is generally consistent with the water system goals established in the ORP.

The ORP aims to improve Oregon's resilience over 50 years from the time the plan was issued in 2013. Between now and 2063, JWC will leverage this Plan to continue to work toward increasing the resilience of their water system through prioritized retrofits and replacements.

3.2.1 Water System Backbone

A water system's backbone consists of its critical components including supply sources, treatment facilities, transmission mains, pump stations, reservoirs, and distribution pipes that must be resilient enough to operate and serve critical customers who are required to meet short-term community needs shortly after a major earthquake. As shown in Figure 3.1, all of JWC's water system assets are considered to be part of the backbone.

Since implementing significant repairs to the backbone system would be difficult in the initial days following a major earthquake, elements must be designed or retrofitted so they experience only minor or no geotechnical, structural, and non-structural damage during the event. JWC's LOS goals necessitate backbone facilities meet the immediate occupancy structural performance objective and operational non-structural performance objective, both of which are described in Section 3.2.3. If seismic assessment of the backbone components reveals gaps between the existing structures' expected performance and recommended performance objectives, deficient components must be retrofitted or replaced, as appropriate.

3.2.2 Post-Earthquake Level of Service Goals

JWC previously developed post-earthquake LOS goals for the three earthquake hazard levels listed in Table 3.18, which have been summarized from the *2008 Water Treatment Plant Seismic Evaluation* and *2018 Water Treatment Plant Facility Plan*. These LOS goals were established prior to development of the ORP.

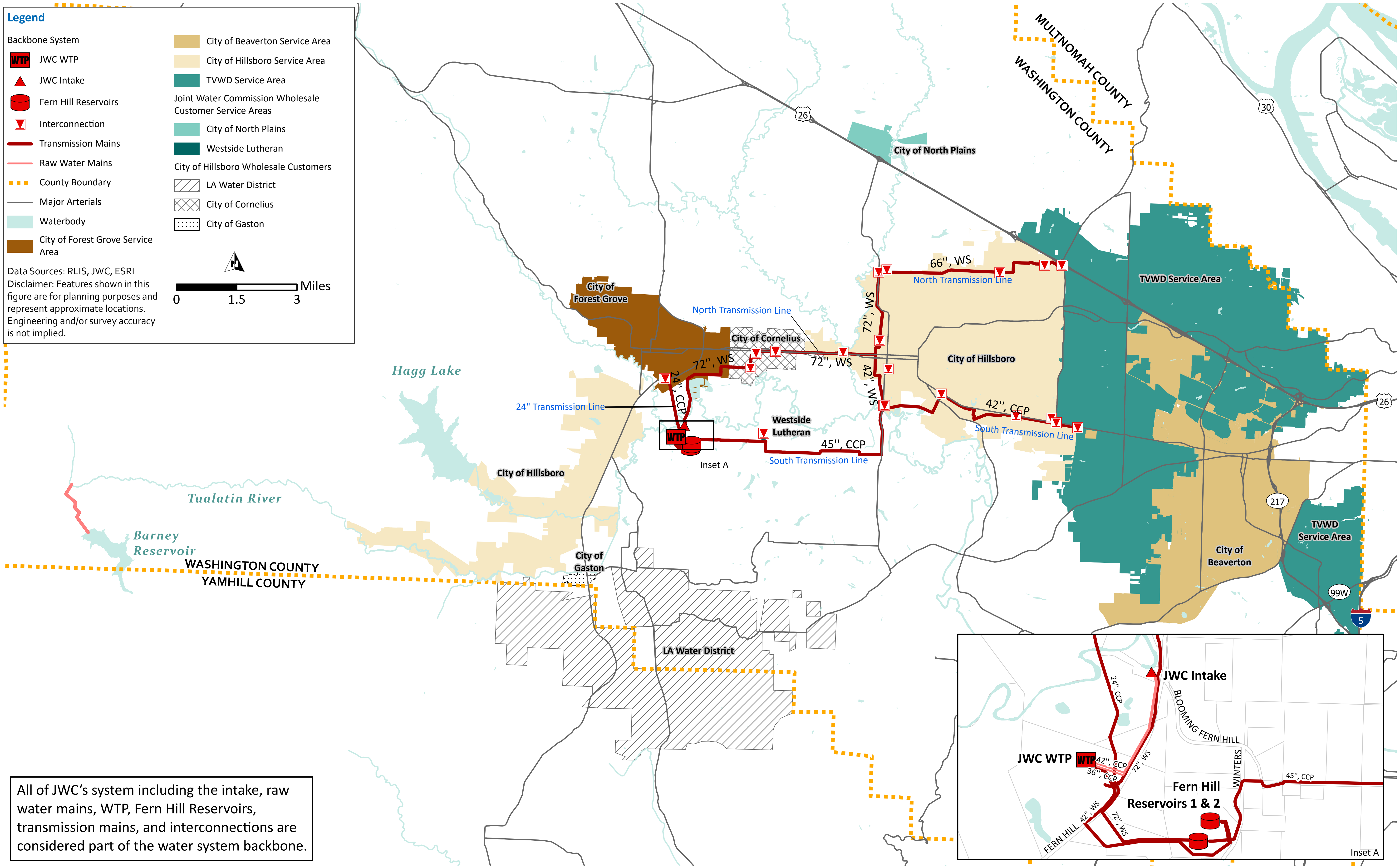
Table 3.18 Existing JWC Post-Earthquake LOS Goals

Seismic Event	Immediate Capacity (mgd)	Short-Term Capacity (mgd)	Short-Term Restoration Time (Days)	Water Quality
72-year event	42 (ADD)	42 (ADD)	0	Potable
M9.0 CSZ event	0	28 (WADD)	1	Potable
2,475-year event, by day 3 capacity	0	14 (1/2 WADD)	3	Potable
2,475-year event, by day 14 capacity	0	28 (WADD)	7 to 14	Potable
2,475-year event, by day 90 capacity	0	42 (ADD)	60 to 90	Potable

Notes:
Abbreviations: ADD - Average Day Demand; WADD - Winter Average Day Demand.
(1) LOS goals adapted from 2008 Water Treatment Plant Seismic Evaluation and 2018 Water Treatment Plant Facility Plan.

---

[2] *Big Steps Before the Big One: How the Portland area can bounce back after a major earthquake*, City Club of Portland, Portland, OR.



All of JWC's system including the intake, raw water mains, WTP, Fern Hill Reservoirs, transmission mains, and interconnections are considered part of the water system backbone.

Figure 3.1 Water System Backbone

The ORP recommends a three-tiered approach to defining the speed of recovery and implementing phased restoration of services for a community's infrastructure system following a M9.0 CSZ earthquake. The three tiers are 30 percent operational, 60 percent operational, and 90 percent operational. The ORP recommends a timeline for these three-tiered LOS goals; however, this recommendation also has built-in flexibility so that a utility can define ways to achieve levels of functional restoration that are specific to their system.

3.2.2.1 Recommended Water Quantity Level of Service Goals

The post-earthquake LOS goals (i.e., restoration timeline) that have been recommended for JWC's adoption align with their existing goals as well as those documented in the ORP and are augmented by additional considerations suggested by the National Institute of Standards and Technology's (NIST's) *Community Resilience Planning Guide for Buildings and Infrastructure Systems* (NIST. 2016)[3]. Table 3.19 summarizes recommended LOS goals following an M9.0 CSZ earthquake while Table 3.20 summarizes recommended goals following an earthquake with a 2,475-year return period.

Table 3.19 and Table 3.20 also provide additional information in defining what 30, 60, and 90 percent operational means for JWC's water system infrastructure. For instance, the 90 percent operational goal for backbone transmission facilities (i.e., pipelines, pump stations, and tanks) means that transmission system must be capable of delivering 90 percent of the partner agencies' 2063 average annual daily demand from the JWC water system. Year 2063's demands were selected to align with the 50-year ORP target year for achieving state-wide resilience goals.

---

[3] *Community Resilience Planning Guide for Building and Infrastructure Systems*, NIST Special Publication 1190, National Institute of Standards and Technology, Gaithersburg, MD.

Table 3.19 Recommended JWC Water System Recovery Goals – following an M9.0 CSZ Earthquake (adapted from OSSPAC 2013 and NIST 2016)

Water Systems	Target Timeframe for Recovery							
	Phase 1: Short-Term (Days)			Phase 2: Intermediate (Weeks)			Phase 3: Long-Term (Months)	
	0-1	1-3	3-7	1-2	2-4	4-12	3-6	6-12
Source								
Raw water reservoirs	30% of 2063 ADD	60% of 2063 ADD		90% of 2063 ADD				
Raw water intake and pump station	30% of 2063 ADD	60% of 2063 ADD		90% of 2063 ADD				
Raw water conveyance piping (Barney Reservoir and Intake to WTP)	90% of 2063 ADD							
Water Production	30% of 2063 ADD	60% of 2063 ADD		90% of 2063 ADD				
Water Quality								
Treatment operations functional(1)	30% of 2063 ADD (Potable)	60% of 2063 ADD (Potable)		90% of 2063 ADD				
Transmission								
Backbone transmission facilities (pipelines, pump station, and tanks)	90% of 2063 ADD							
Control Systems								
SCADA and other control systems	90% of components required for normal operation are functional							

Note:
(1) JWC's goal is to deliver potable water that meets the minimum Oregon Health Authority requirements immediately following the event.

Key to Table:

Desired time to restore components to 30 percent operational (Red)	R
Desired time to restore components to 60 percent operational (Yellow)	Y
Desired time to restore components to 90 percent operational (Green)	G

Table 3.20 Recommended JWC Water System Recovery Goals – following a 2,475 year Return Period Earthquake (adapted from OSSPAC 2013 and NIST 2016)

Water Systems	Target Timeframe for Recovery							
	Phase 1: Short-Term (Days)			Phase 2: Intermediate (Weeks)			Phase 3: Long-Term (Months)	
	0-1	1-3	3-7	1-2	2-4	4-12	3-6	6-12
Source								
Raw water reservoirs		30% of 2063 ADD		60% of 2063 ADD		90% of 2063 ADD		
Raw water intake and pump station		30% of 2063 ADD		60% of 2063 ADD		90% of 2063 ADD		
Raw water conveyance piping (Barney Reservoir and Intake to WTP)	90% of 2063 ADD							
Water Production		30% of 2063 ADD		60% of 2063 ADD		90% of 2063 ADD		
Water Quality								
Treatment operations functional(1)		30% of 2063 ADD (Potable)		60% of 2063 ADD (Potable)		90% of 2063 ADD (Potable)		
Transmission								
Backbone transmission facilities (pipelines, pump station, and tanks)	90% of 2063 WDD							
Control Systems								
SCADA and other control systems	90% of components required for normal operation are functional							

Notes:
(1) JWC's goal is to deliver potable water that meets the minimum Oregon Health Authority requirements immediately following the event.

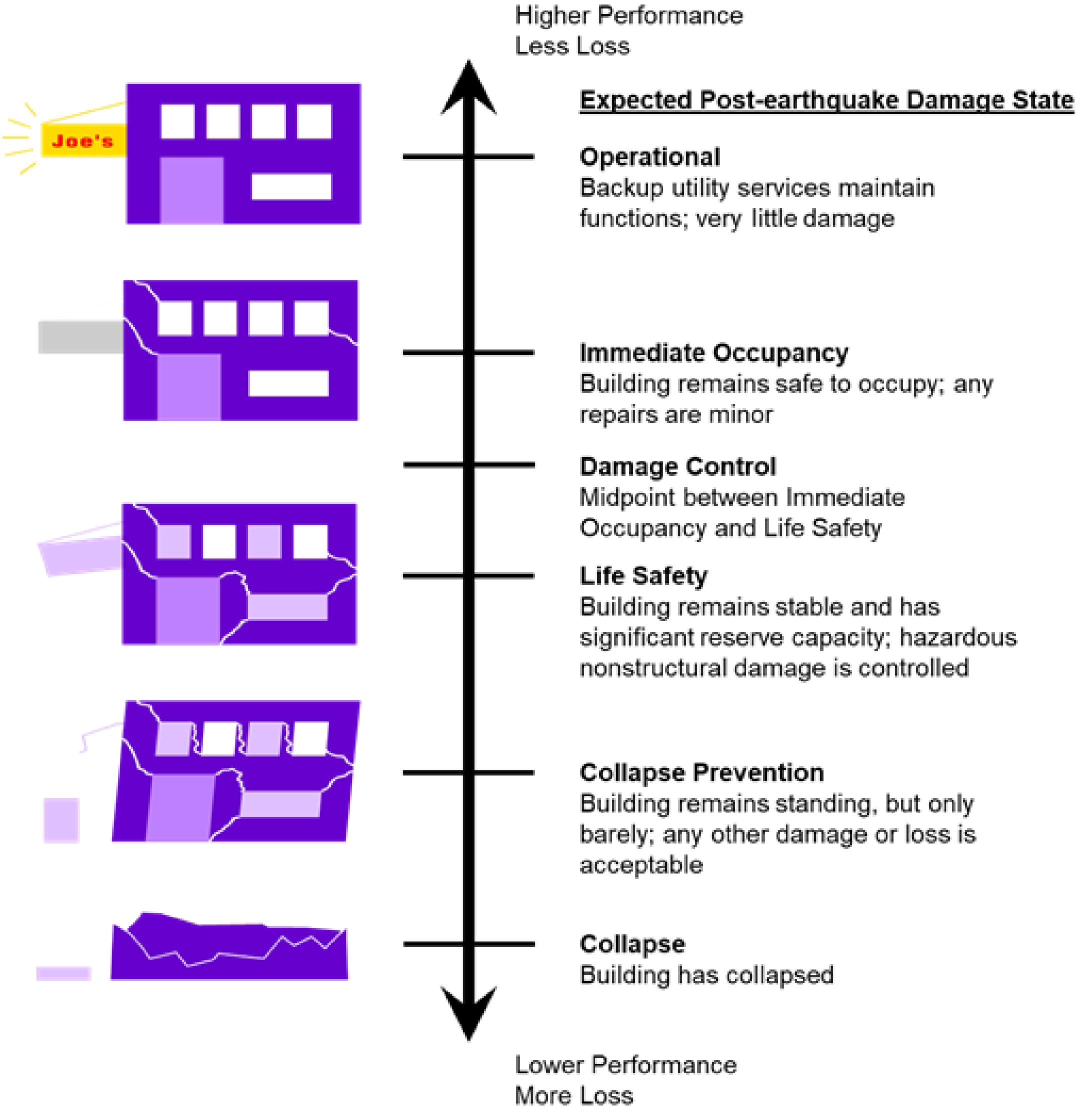
Key to Table:

Desired time to restore components to 30 percent operational (Red)	R
Desired time to restore components to 60 percent operational (Yellow)	Y
Desired time to restore components to 90 percent operational (Green)	G

3.2.3 Seismic Performance Objectives

To meet JWC's post-earthquake LOS goals, water system structures must meet or exceed defined levels of seismic performance. American Society of Civil Engineers (ASCE) 41-17, *Seismic Evaluation and Retrofit of Existing Buildings* (2017b)[4], presents several structural and non-structural seismic performance levels and the expected level of earthquake damage associated with each. Also included are expectations regarding the operability and reparability of buildings that suffer earthquake damage under the various performance levels.

Figure 3.2 illustrates ASCE 41-17's performance levels.



Expected Post-earthquake Damage State

Operational
Backup utility services maintain functions; very little damage

Immediate Occupancy
Building remains safe to occupy; any repairs are minor

Damage Control
Midpoint between Immediate Occupancy and Life Safety

Life Safety
Building remains stable and has significant reserve capacity; hazardous nonstructural damage is controlled

Collapse Prevention
Building remains standing, but only barely; any other damage or loss is acceptable

Collapse
Building has collapsed

Figure 3.2 Building Performance Levels (adapted from ASCE, 2017b)

---

[4] ASCE 41-17, *Seismic Evaluation and Retrofit of Existing Buildings*, American Society of Civil Engineers, Reston, VA.

According to the latest edition of the Oregon Structural Specialty Code (OSSC) (2019)[5], water storage facilities and pump stations that must maintain water pressure for fire suppression are designated as Risk Category IV structures. Conversely, water system structures not required to maintain water pressure for fire suppression are designated as Risk Category III structures. Table 3.21 compares ASCE 41-17's performance objectives against the expected performance associated with OSSC's risk categories.

Table 3.21 Comparison of Seismic Performance Objectives with OSSC Risk Categories

Risk Category	Structural Performance Objective(1)	Non-structural Performance Objectives(1)
IV	Immediate Occupancy	Operational
III	Damage Control	Position Retention
I & II	Life Safety	Position Retention

Note:

(1) For the Design Earthquake (2/3 of the Risk-Targeted Maximum Considered Earthquake [MCE_R]) as defined by the 2019 OSSC or Basic Safety Earthquake-1 (BSE-1N) seismic hazard level as defined by ASCE 41-17.

3.2.3.1 Structural Performance Levels

ASCE 41-17 specifies the following three structural performance levels:

- **Immediate Occupancy**: This performance level refers to the post-earthquake damage state in which only minimal structural damage occurred. Under this category, a building's basic vertical and lateral force-resisting systems retain almost all their pre-earthquake strength and stiffness, and the risk of life-threatening injury from structural damage is very low. Although some minor structural repairs may be appropriate, they are generally not required before re-occupancy. Finally, the building's structural condition does not limit its continued use; however, damage or disruption to non-structural elements of the building, furnishings, or equipment, as well as the availability of external utility services, may be limiting.
- **Damage Control**: This performance level refers to a midway point between Life Safety and Immediate Occupancy. The goal of this category is to provide a structure that can more reliably resist collapse and incur less damage than a typical structure but not to the extent required of a structure designed to meet Immediate Occupancy. Being in the middle of both levels, Damage Control provides a greater margin of safety against collapse than Life Safety, allowing occupants to return to function more quickly but not as quickly as Immediate Occupancy.
- **Life Safety**: This performance level refers to the post-earthquake damage state in which a structure has been significantly damaged, but some margin against either partial or total structural collapse remains. Under this category, some structural elements and components are severely damaged, but this damage has not caused large falling debris hazards inside or outside a building. Injuries may occur during the earthquake, but the overall risk of life-threatening injury from structural damage is expectedly low. Finally, repairing the structure should be possible but may not be practical for economic reasons; although the damaged structure is not an imminent collapse risk, implementing structural repairs or installing temporary bracing before re-occupancy would be prudent.

---

[5] *Oregon Structural Specialty Code*, International Code Council, Country Club Hills, IL.

3.2.3.2 Non-structural Performance Levels

In addition to structural performance levels, ASCE 41-17 specifies the following two non-structural performance levels:

- **Operational**: This performance level describes a post-earthquake situation in which most non-structural systems required for a building's normal use are functional, though minor cleanup and repairs may be required. Achieving this level requires considering many elements beyond those normally within a structural engineer's responsibilities. In addition to ensuring that non-structural components are properly mounted and braced within the structure, emergency standby equipment is often needed to provide utility services from external sources that might be disrupted. Qualification testing may also be required to ensure that all necessary equipment functions during or after strong shaking.
- **Position Retention**: This performance level refers to non-structural conditions in which, presuming the building is structurally safe after an earthquake, occupants can re-occupy safely with some limitations. Normal use might be impaired, some cleanup might be needed, and some inspection might be warranted; however, building equipment is generally secured in place and may function if the necessary utility services are available. Some components may experience misalignments or internal damage and, thus, be rendered inoperable.
  At this performance level, power, water, natural gas, communications lines, and other utilities required for normal building use may be unavailable. Cladding, glazing, ceilings, and partitions might be damaged but do not present safety hazards or conditions preventing occupation. The risk of life-threatening injury caused by non-structural damage is also very low.

3.2.3.3 Recommended Performance Objectives

To achieve the recommended LOS goals, the water system must be restored to a 90 percent operational level within 1 to 2 weeks after an M9.0 CSZ earthquake and 4 to 12 weeks after an earthquake with a 2,475-year return period. These restoration timelines suggest that most system components must achieve the Immediate Occupancy structural performance level and Operational non-structural performance level for an earthquake with a 2,475-year return period. Detailed geotechnical and structural seismic evaluations should be conducted for existing backbone water system structures, to determine if the recommended LOS goals can be achieved given the current anticipated seismic performance of these structures.

Table 3.22 provides the recommended performance objectives are driven by the performance required to meet the LOS goals for a 2,475-year return period earthquake, the Basic Safety Earthquake-2 (BSE-2N) seismic hazard level defined by ASCE 41-17. These performance objectives for the BSE-2N are to be used with ASCE 41-17's Basic Performance Objective Equivalent to New Building Standards. This BSE-2N seismic hazard level is consistent with the design earthquake defined in the WWSS's *Seismic Guidelines and Minimum Design Requirements* (2020)[6].

---

[6] *Seismic Guidelines and Minimum Design Requirements*, Willamette Water Supply Program, Beaverton, OR.

Table 3.22 Water System Seismic Retrofit Performance Objectives

Restoration Timeline	Retrofit Structural Performance Objective(1)	Retrofit Non-structural Performance Objective(1)
Days to Weeks	Immediate Occupancy	Operational
Weeks to Months	Immediate Occupancy	Operational

Note:
(1) For the BSE-2N seismic hazard level as defined by ASCE 41-17.

For new structures, the increased construction cost of elevating the design standard from Risk Category III to Risk Category IV is typically relatively minor. Therefore, all new water system structures are recommended to be designed against more stringent OSSC seismic requirements for Risk Category IV structures, and the design earthquake is recommended to be redefined as a 2,475-year return-period event, which ASCE 7-16 calls a Risk-Targeted Maximum Considered Earthquake.

Additionally, because geotechnical hazards (e.g., liquefaction and lateral spreading) can significantly affect the performance of water system structures after a major earthquake, conducting site-specific geotechnical investigations and analyses is recommended to characterize these potential hazards. Beyond that, water system structure designs must include appropriate measures to mitigate these potential site-specific geotechnical hazards: Per the requirements of the latest edition of ASCE 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (2017a)[7], equipment associated with water system structures must be adequately braced and seismically certified, allowing equipment to remain operational after a design-level earthquake as long as dependent systems (e.g., electrical power) are also functional. Piping entering or exiting water system structures must also be designed to accommodate the anticipated earthquake-induced relative movement between the structure and surrounding soil.

As discussed in Section 3.1.6, JWC is recommended to develop seismic design standards to guide seismic design and retrofit guidelines for their facilities. WWSS's *Seismic Guidelines and Minimum Design Requirements* may be adopted with appropriate modifications made where necessary.

3.2.4 Water Quality Level of Service Goals

Whether faced with an M9.0 CSZ earthquake or an earthquake with a 2,475-year return period, JWC's goal is to deliver potable water that meets the Oregon Health Authority's (OHA's) requirements to the partners following the event.

3.2.5 Other Potential Natural Hazards

In addition to earthquake hazards, the JWC water system may be affected by other natural hazards including, but not limited to the following:

- Wildfire.
- Flooding (including consideration for the potential impacts of climate change).
- Hazardous algal blooms.
- Drought (including consideration for the potential impacts of climate change).

---

[7] ASCE 7-16, *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, Reston, VA.

It is recommended that JWC consider adopting LOS goals for these additional hazards that may potentially impact the JWC system. The following are example LOS goals to use as a starting point:

- For water quality-impacting events including wildfire, flooding, and harmful algal blooms, the immediate and short-term WTP capacity will be the winter average day demand with potable water quality.
- For drought events, the WTP will supply the average day demand with potable water quality throughout the duration of the event.

It is recommended that JWC conduct additional assessments to characterize the system's expected performance for these listed hazards and develop mitigation concepts, as required, to help ensure that JWC can meet the recommended LOS goals.

3.3 Recommended Further Study

To summarize, the following additional studies or work efforts are recommended to supplement JWC's existing policies and LOS goals. These recommendations encompass industry best practices and matters of interest to the public:

- Perform a hazard mitigation study to include the following components:
    - Review and confirm earthquake-related LOS goals considering regional resilient water needs and supplies:
        - Adopt earthquake-related LOS goals to align with the ORP's three-tiered restoration approach.
    - Adopt LOS goals for water-quality-impacting events and drought events and conduct additional assessments of the system to characterize performance and develop mitigation concepts to ensure that the system can effectively meet LOS goals.
    - Conduct additional studies to determine potential for landslide-induced blockages of the Tualatin River that would inhibit the JWC WTP's raw water supply.
    - Conduct WTP non-structural seismic evaluations. For WTP facilities that are expected to structurally withstand earthquakes, perform additional assessments to determine if the existing facilities will meet the Operational non-structural performance objectives:
        - Evaluate ruggedness of equipment and nonstructural components.
        - Review pipeline-structure interface connections throughout the facility to confirm if flexible joints are present to accommodate expected differential settlement.
        - Investigate effects of potential interactions between adjacent process units connected by large diameter process pipelines.
        - Develop mitigation concepts to ensure the system can effectively meet the established LOS goals and recommended performance objectives.
    - Conduct assessments (e.g., geotechnical and structural seismic) of the Fern Hill Reservoirs to characterize expected performance for given hazards and develop mitigation concepts to ensure that the system can effectively meet the established LOS goals and recommended performance objectives. The resilience evaluation

  should also include the following to determine if the reservoirs and associated facilities will meet the Operational non-structural performance objective:
    - Evaluate ruggedness of equipment and nonstructural components.
    - Review pipeline-structure interface connections throughout the facility to confirm if flexible joints are present to accommodate expected differential settlement.
- Develop an emergency water supply strategy that can be implemented before major seismic upgrades to vulnerable JWC facilities are completed:
    - Evaluate spare parts needs and improvements to meet established goals.
- Identify and assess critical system and organizational dependencies in the aftermath of a seismic event and other hazard events. Review the Emergency Response and Continuity of Operations Plan to identify potential gaps and dependencies that would delay functional recovery of the water system and develop recommendations to help achieve the established LOS goals.
- Develop seismic design standards that are consistent with adopted LOS goals to guide seismic design and retrofit guidelines for JWC facilities and pipelines, taking into consideration current national seismic policies (e.g. functional recovery of critical infrastructure systems) and the design guidelines implemented by major West Coast water agencies (including Los Angeles, San Francisco, Seattle, and WWSS).

Chapter 4

WATER QUALITY AND REGULATORY FRAMEWORK

4.1 Introduction

The Joint Water Commission's (JWC) Water Treatment Plant (WTP) withdraws raw water from the Tualatin River through the intake structure at the Spring Hill Pumping Plant and delivers finished water to the four partners: The Cities of Beaverton, Forest Grove, and Hillsboro and the Tualatin Valley Water District (TVWD).

This chapter discusses the following topics surrounding JWC's water quality and regulatory framework:

- JWC's current regulatory compliance status.
- Historical data for the raw water quality from the Tualatin River, the finished water quality leaving the JWC WTP, and select distribution system water quality for the JWC partners.
- Contaminants of emerging concern (CECs).
- Anticipated future regulatory requirements.
- WTP process capabilities in addressing anticipated water quality challenges.
- Finished water quality goals.

4.2 Water Quality and Regulatory Compliance

This section summarizes JWC's historical water quality data and compliance with existing regulatory requirements for water quality.

4.2.1 Regulatory Compliance

The JWC WTP's finished water must comply with the following current federal drinking water regulations:

- National Primary Drinking Water Regulations (1975).
- National Secondary Drinking Water Regulations (1979, 1991).
- Phase I, II, and V Regulations for Inorganic Contaminants, Synthetic Organic Contaminants, and Volatile Organic Contaminants (1987, 1991, and 1992, respectively).
- Surface Water Treatment Rule (1989).
- Interim Enhanced Surface Water Treatment Rule (1999).
- Total Coliform Rule (1989).
- Lead and Copper Rule (1991).
- Consumer Confidence Reports Rule (1998).
- Stage 1 Disinfectants and Disinfection Byproduct Rule (Stage 1 DBPR) (1998).
- Stage 2 Disinfectants and Disinfection Byproduct Rule (Stage 2 DBPR) (2006).
- Unregulated Contaminant Monitoring Rule 1 (1999), 2 (2007), 3 (2012), and 4 (2016).

- Long-Term Stage 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) (2006).
- Radionuclides Rule (2000).
- Arsenic Rule (2001).
- Filter Backwash Recycle Rule (2001).

As illustrated in the water quality tables in Section 4.2.3 through Section 4.2.8, all contaminants that were detected in the WTP's finished water are below their respective maximum contaminant levels (MCLs) set by the U.S Environmental Protection Agency (EPA) and by the Oregon Health Authority (OHA) in Oregon Administrative Rule (OAR), Chapter 333, Division 061. As such, JWC continues to fulfill their established goal of supplying water that meets or exceeds all regulatory requirements.

4.2.2 Water Quality Sampling

Enacted in 1981, the Oregon Drinking Water Quality Act, including Oregon Revised Statute (ORS) 448.119 to 448.285, 454.235, and 454.255, has been periodically amended. Per the OHA, the act aims to ensure that all Oregonians have safe drinking water, provide a simple and effective regulatory program for drinking water systems, and establish a means to improve inadequate systems.

ORS 448.131 authorizes the OHA to adopt administrative rules to ensure safe drinking water is provided to the public, and OAR, Chapter 333, Division 061, is reserved for regulations placed on public water systems. Table 4.1 and Table 4.2 summarize JWC's current OHA sampling requirements for their raw and finished water, respectively. Each JWC partner individually completes the required sampling for water quality parameters in their distribution systems.

Table 4.1 Raw Water Quality Required Sampling Schedule

Contaminant Group	Regulatory Required Sampling Interval(1)	Notes
TOC	Quarterly	
Alkalinity	Quarterly	
Cyanotoxins(2)	Biweekly	Sampling for total microcystins and cylindrospermopsin required in source water from May 1 to October 30.

Notes:
Abbreviation: TOC – total organic carbon.
(1) Required sampling schedule from OHA Drinking Water Data Online as of August 2021.
(2) Permanent cyanotoxin monitoring rules for total microcystins and cylindrospermopsin became effective at the end of 2018. Additional cyanotoxin-related sampling requirements, beyond those shown, are triggered based on detected cyanotoxin concentrations in the raw water and/or recreational use health advisories in source waters.

Table 4.2 Finished Water Quality Required Sampling Schedule

Contaminant Group	Regulatory Required Sampling Interval(1)	Notes
Arsenic	9 years	
Hexachlorocyclopentadiene	Yearly	
IOCs	9 years	
Nitrate	Yearly	
Nitrite	9 years	

Contaminant Group	Regulatory Required Sampling Interval(1)	Notes
pH	Daily	
Radionuclides	9 years	
SOCs	3 years	Must be sampled for two consecutive quarters.
VOCs	Yearly	
TOC	Monthly / Quarterly	
Turbidity	Daily	
Total Trihalomethanes(2)	Yearly	Sample must be collected in July
HAA5	Yearly	Sample must be collected in July

Note:
Abbreviations: IOC – inorganic contaminant; SOC – synthetic organic contaminant; VOC – volatile organic contaminant; HAA5 – a group of five haloacetic acids (monochloroacetic acid, dichloroacetic acid, trichloroacetic acid, monobromoacetic acid, and dibromoacetic acid).
(1) Required sampling schedule from OHA Drinking Water Data Online as of August 2021.
(2) Total trihalomethanes includes bromodichloromethane, dibromochloromethane, bromoform, and chloroform.

JWC maintains a robust raw and finished water quality sampling program that exceeds the OHA's requirements, including monitoring at the Barney Reservoir and Hagg Lake. In addition to the required OHA sampling, JWC elects to semiannually test for regulated inorganic contaminants (IOCs), volatile organic contaminants (VOCs), synthetic organic contaminants (SOCs), and additional unregulated compounds in their raw and finished water. JWC also maintains a separate *Algal Bloom Monitoring and Response Strategy* that includes additional monitoring and sampling efforts to anticipate and prepare for an algal event. The algal bloom monitoring strategy includes procedures for routine visual inspections, continuous monitoring from data sondes, and grab sampling to proactively monitor for potential algal events.

To monitor contaminant occurrence and treatment performance, the JWC collects grab samples for a range of water quality constituents at frequent intervals, and, to monitor treatment performance, uses online analyzers and laboratory equipment to measure parameters of particular interest. The JWC also supports multiple water quality monitoring stations in the watershed through joint funding agreements with the United States Geological Survey.

Appendix J presents full sampling results and water quality data for JWC's raw and finished water. Note that this chapter does not review sampling and monitoring data collected as part of algal bloom monitoring efforts.

4.2.3 General Parameters and Secondary Contaminants

Table 4.3 summarizes general raw water quality parameters from the intake on the Tualatin River. The finished water's MCLs or secondary MCLs, which are non-enforceable guidelines set for contaminants that may aesthetically affect the finished water, illustrate where removal or treatment is required.

Table 4.3 Summary of General Raw Water Quality Parameters

Raw Water Parameter	Unit	Finished Water MCL	No. of Samples	Value Range	Average
Temperature (1)	degrees C	None	1,303	3.1 – 17.8(2)	10.8(3)
pH(1)		7.2(7)	1,304	6.4 – 7.6(2)	7.0(3)
Turbidity(1)	NTU	Treatment technique. ≤0.3 NTU for 95% of samples. <1.0 at all times	1,891	2.4 – 138.8(2)	10.0(3)
TOC(1)	mg/L	None	1,860	0.55 – 9.99(2)	1.67(3)
DOC(4)	mg/L	None	13	0.96 – 2.35(5)	1.49
Alkalinity(6)	mg/L as $CaCO_3$	None	76	17.1 – 40.0(5)	29.9
Hardness(4)	mg/L as $CaCO_3$	250	13	23.5 – 41.8(5)	28.8

Notes:

Abbreviations: C – degrees Celsius; NTU – nephelometric turbidity unit; DOC – dissolved organic carbon; mg/L – milligrams per liter; $CaCO_3$ - calcium carbonate.

(1) Data reported for daily maximum, minimum, and average values for raw water TOC, pH, and temperature from February 2016 to April 2021. pH data not available prior to August 2017.

(2) Range is minimum to maximum of daily average values.

(3) Average of all daily averages recorded in daily monitoring data.

(4) From semiannual grab sampling of raw water between 2016 and 2021.

(5) Value range is minimum to maximum of all samples.

(6) From periodic raw water grab sampling between 2016 and 2021.

(7) Finished water pH target set by OHA for corrosion control.

The average raw water turbidity entering the JWC WTP was 10 nephelometric turbidity units (NTU), with noticeable variations throughout the year, particularly during the winter and spring months. Figure 4.1 shows the monthly average turbidity with error bars providing the 5th and 95th percentile turbidity. Higher-turbidity values correspond with increased precipitation and watershed flushing that occur during the winter and spring runoff seasons.

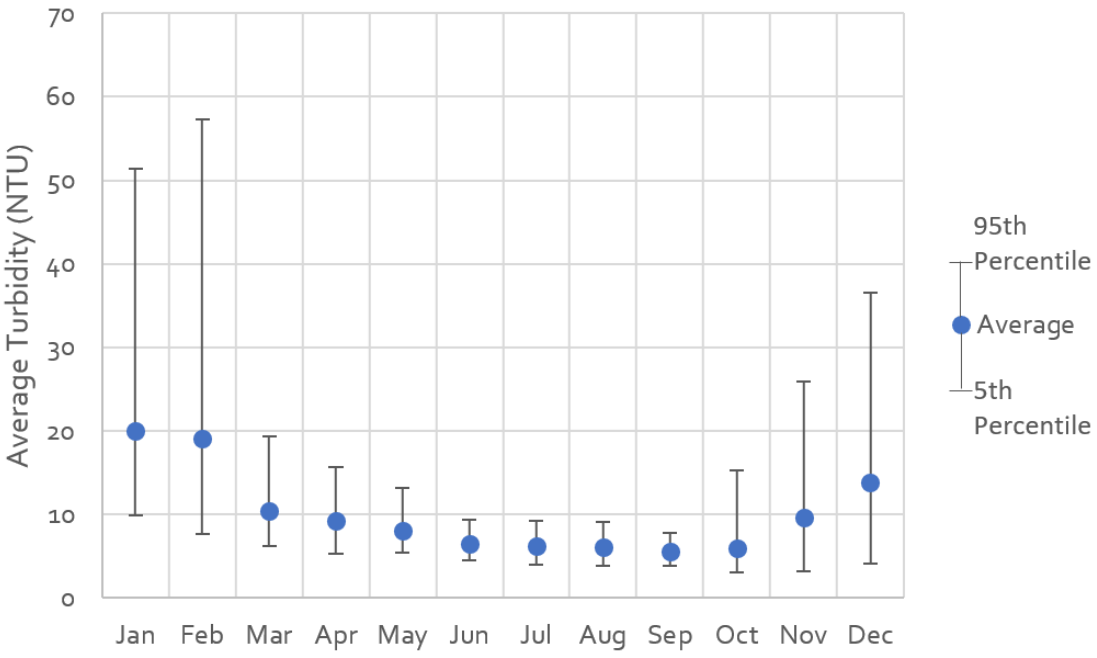


Figure 4.1 Monthly Average Raw Water Turbidity (February 2016 to April 2021)

The average raw water alkalinity was 29.9 milligrams per liter (mg/L); the Tualatin River is considered a low alkalinity water supply source.

On average, TOC in the raw water was 1.67 mg/L, approximately 80 percent of 2 mg/L (the concentration that triggers removal for regulatory compliance), with exceedances of this threshold at certain times of the year. Figure 4.2 shows the seasonal variability is generally consistent with seasonal turbidity patterns, with the highest TOC and greatest variability occurring between October and April. According to limited sampling of dissolved organic carbon (DOC) and daily sampling of TOC, almost 90 percent of the total TOC is present in a dissolved form.

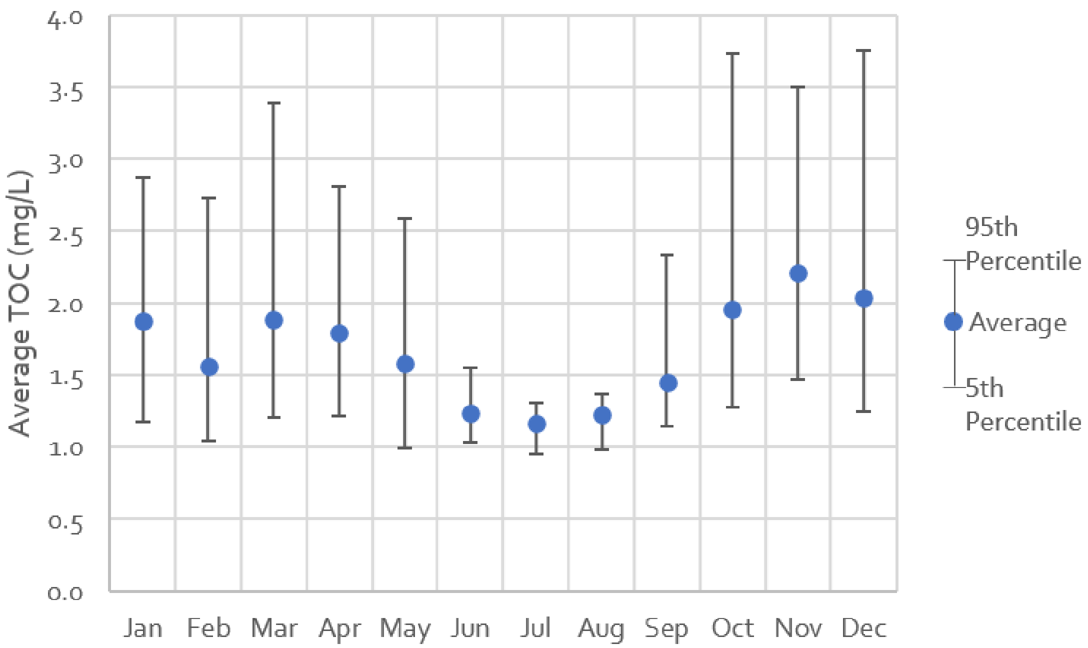


Figure 4.2 Monthly Average Raw Water TOC (February 2016 to April 2021)

Table 4.4 summarizes general finished water quality parameters. For the period examined, the average combined filter effluent (CFE) turbidity was 0.03 NTU, an order of magnitude below the regulatory requirement of less than 0.3 NTU 95 percent of the time and below 1.0 NTU at all times. This data indicates excellent treatment performance, even during periods of elevated raw water turbidity.

With the addition of filters 15 and 16 as part of the expansion to 85 million gallons per day (mgd) JWC has two regulatory compliance points, one for the CFE from filters 1 - 14 and one for the CFE from filters 15 and 16. Note that the data presented in Table 4.4 only include CFE turbidity data for filters 1 - 14 due to limited data availability for filters 15 and 16 CFE for the period examined. JWC reports the higher of the two combined filter effluent turbidity values for regulatory compliance.

Table 4.4 Summary of General Finished Water Quality Parameters

Finished Water Parameter	Unit	Finished Water MCL	No. of Samples	Value Range	Average
Temperature(1)	degrees C	None	1,216	2.3 – 20.9(2)	10.7(3)
pH(1)		7.2(8)	1,878	7.1 – 8.0(2)	7.71(3)
Turbidity – CFE(1)(9)	NTU	Treatment technique. ≤0.3 NTU for 95% of samples. <1.0 at all times	1,891	0.01 – 0.24(2)(9)	0.03(3)
TOC(1)	mg/L	None	1,860	0.30 – 1.76(2)	0.78(3)
DOC(4)	mg/L	None	10	0.49 – 1.12(5)	0.77

Finished Water Parameter	Unit	Finished Water MCL	No. of Samples	Value Range	Average
Alkalinity(6)	mg/L as $CaCO_3$	None	3,768	24 – 38(7)	30
Hardness(4)	mg/L as $CaCO_3$	250	38	21.7 – 35.6(5)	29.4

Notes:

(1) Data reported for daily maximum, minimum, and average values for finished water TOC, pH, and temperature from February 2016 to April 2021. Temperature data not available prior to January 2017.
(2) Range is minimum to maximum of daily average values.
(3) Average of all daily average recorded in daily monitoring data.
(4) From semiannual grab sampling of finished water between 2016 and 2021.
(5) Value range is minimum to maximum of all samples.
(6) From daily measurements of alkalinity January 2016–April 2021. Alkalinity measured twice daily, once in the am and pm.
(7) Range is 5th percentile to 95th percentile.
(8) Finished water pH target set by OHA for corrosion control.
(9) Combined filter effluent turbidity data from filters 1 – 14. Combined filter effluent turbidity data for filters 15/16 not included due to limited data availability for the period examined.

As is discussed later in Section 4.5, JWC's goal is to provide 35 to 45 percent removal of TOC even when raw water TOC is below concentrations that require a specific percentage removal. The average TOC in the finished water was 0.78 mg/L, indicating that more than 50 percent of removal is achieved on average, exceeding JWC's goal. Figure 4.3 shows average monthly TOC removal rates alongside the average monthly raw and finished water TOC.

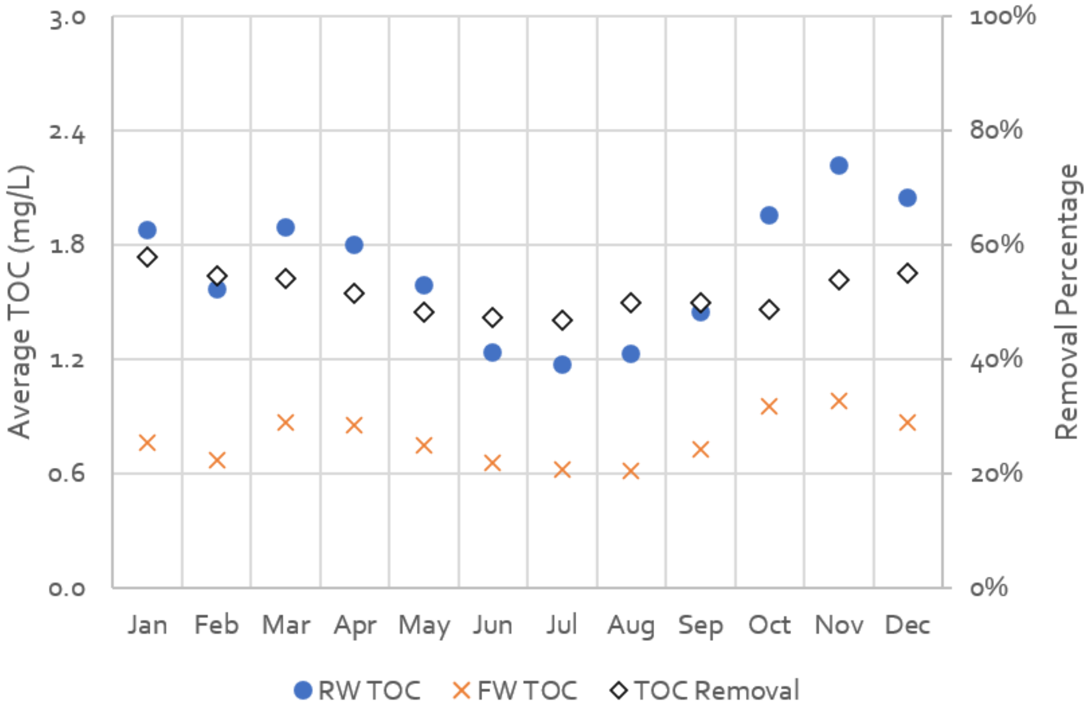


Figure 4.3 Monthly Average TOC Removal

Average TOC removal varies between 45 and 60 percent, with the higher removal percentages observed during periods with higher TOC. Averaging from 0.7 to 1.0 mg/L year-round, finished water TOC was less variable than raw water TOC, which ranged between 1.2 and 2.2 mg/L.

To meet the OHA's requirements for corrosion control in distribution systems, the daily average finished water pH must be 7.2 or higher. An excursion occurs when the daily average value of the finished water pH falls below 7.2; JWC can have no more than 9 excursions in a 6-month period to comply with OHA's requirements.

Finished water pH for JWC is calculated differently, depending on which finished water pump stations are online. When both finished water pump stations 1 and 2 are online, finished water pH is taken as the average pH from both pump stations, with values taken every 2 hours to calculate the daily average. If only finished water pump station 1 is online, the daily average finished water pH is calculated as the average of values taken every 2 hours from finished water pump station 1.

The average finished water pH was 7.7, consistent with JWC's target pH of 7.7. JWC had a single pH excursion in December 2018. No excursions have occurred since.

4.2.4 Secondary Contaminants

JWC has conducted multiple evaluations related to the presence of metals in their raw and finished water, primarily focusing on iron and manganese after an elevated turbidity event that occurred in the Fern Hill Reservoirs in 2017 after replacement of the filter media. These evaluations identified the presence of a wide variety of trace metals throughout the watershed, though generally at levels below their finished water MCLs.

In response, JWC has increased their sampling efforts for iron and manganese. Table 4.5 summarizes data from this increased sampling along with results from semiannual sampling of aluminum, another metal regulated as a secondary contaminant.

Table 4.5 Summary of Select Raw Water Secondary Contaminants

Raw Water Contaminant	Unit	Finished Water Secondary MCL(4)	No. of Detects	No. of Samples	Value Range(2)	Average
Aluminum(1)	mg/L	0.05 – 0.2(3)	15	15	0.06 – 2.37	0.4
Iron(1)	mg/L	0.3	325	325	0.008 – 4.35	0.3
Manganese(1)	mg/L	0.05	328	329	0.0016 – 0.3	0.019

Notes:
(1) All data from semiannual grab sampling or more frequent grab sampling for iron and manganese between 2016 and 2021.
(2) Value range is minimum to maximum.
(3) EPA established a range with individual states given discretion to establish a specific target. OHA has not established an MCL different from this target range. The lower limit of 0.05 mg/L is an optimal value established by EPA, with the range providing flexibility to account for variability in water quality and treatment processes. Coagulation with aluminum salts is a common source of aluminum in finished water.
(4) Secondary MCLs are non-enforceable guidelines set for contaminants that may aesthetically affect the finished water.

Consistent with the results of previous evaluations, the sampling data shows that JWC's raw water contains low levels of trace metals. Iron has been detected in all samples at an average concentration that is at its secondary MCL. Manganese was consistently detected, although generally at values well below its secondary MCL. Manganese can cause colored water at concentrations as low as 0.02 mg/L.

JWC's previous investigations into the prevalence of metals in raw water also included evaluations of metals removed through the WTP. In general, these investigations showed existing treatment processes substantially and consistently remove iron and manganese to levels well below their secondary MCLs. The only historical issues with iron and manganese removal were related to the 2017 water quality event which was likely caused by replacement of the filter media, as the new media had not yet fully formed the metal oxide coatings that are responsible for much of the plant's iron and manganese removal.

Finished water sampling for secondary contaminants confirms that existing treatment processes are effective in removing trace metals throughout the WTP. As shown in Table 4.6, iron was detected in only half the finished water samples at an average concentration of 0.01 mg/L, well below the 0.3 mg/L concentration detected in the raw water. While manganese was detected in all but one raw water sample, it was only detected in approximately one-third of all finished water samples at concentrations more than an order of magnitude below its secondary MCL, indicating the effective removal of low levels of manganese entering the plant.

Average finished water concentrations for iron and manganese were both below the finished water quality goals of 0.1 mg/L and 0.02 mg/L, respectively, which were established in the *Iron and Manganese from JWC Supply TM* (Jacobs, 2018). Only one sample exceeded the goal for iron while no samples for manganese exceeded the goal.

Table 4.6 presents several other finished water quality secondary contaminants, with the secondary MCLs shown to provide context for the levels observed in the finished water.

Table 4.6 Summary of Select Finished Water Secondary Contaminants

Finished Water Contaminant	Unit	Finished Water Secondary MCL(6)	No. of Detects	No. of Samples	Value Range	Average
Aluminum(1)	mg/L	0.05 – 0.2(3)	501	519	0.002 – 0.026(4)	0.010
Color(1)	Color Units	15	202	523	1.0 – 8.8(4)	2.9
Chloride(2)	mg/L	250	35	36	4.0 – 6.9(5)	5.3
Iron(2)	mg/L	0.3	117	239	0.001 – 0.150(5)	0.014
Manganese(2)	mg/L	0.05	79	257	0.0001 – 0.0160(5)	0.0025
Odor(2)	TON	3	33	36	1.0 – 10.0(5)	4.4
Sulfate(2)	mg/L	250	35	35	9.6 – 20.6(5)	12.5

Notes:
Abbreviation: TON - threshold odor number.
(1) From daily data, with aluminum and apparent color sampled multiple times per week.
(2) Data from routine grab sampling between 2016 and 2021.
(3) EPA established a range with individual states given discretion to establish a specific target. OHA has not established an MCL different from this target range. The lower limit of 0.05 mg/L is an optimal value established by EPA, with the range providing flexibility to account for variability in water quality and treatment processes. Coagulation with aluminum salts is a common source of aluminum in finished water.
(4) Value range is 5th percentile to 95th percentile.
(5) Value range is minimum to maximum.
(6) Secondary MCLs are non-enforceable guidelines set for contaminants that may aesthetically affect the finished water.

An indicator of galvanic corrosion, chloride-to-sulfate mass ratio (CSMR) values greater than 0.6 are associated with high corrosion of lead solder. However, JWC's average CSMR for the sampling period analyzed was 0.4, which falls below levels indicative of the increased potential for corrosion. Furthermore, chloride concentrations were low, less than 10 mg/L, further limiting susceptibility to galvanic corrosion.

Odor in the finished water, as measured by threshold odor number (TON), averaged 4.4 TON from 2016 to 2021, slightly higher than the secondary MCL of 3 TON. The raw water average for odor was less than the MCL, at 2.9 TON. The increase in observed odor from raw to finished water is likely due to the use of chlorine gas for disinfection.

4.2.5 IOCs, SOCs, and VOCs

At a minimum, JWC semiannually collects grab samples to test for IOCs, VOCs, and SOCs in the raw and finished water. A regulated SOC, atrazine, was detected once in the raw water at a level well below its MCL, with no detections of VOCs. In general, all raw water detections of inorganic contaminants were either well below their MCLs or are currently unregulated. Appendix J presents the full raw water sampling data.

As for the WTP's finished water, a regulated SOC, hexachlorocyclopentadiene, was detected once at a level well below its MCL, with no detections of any regulated VOCs. Additionally, two unregulated VOCs, bromodichloromethane and chloroform, have been frequently detected. Both of these contaminants are trihalomethanes, which are common disinfection byproducts (DBPs) associated with the use of chlorine. While neither is regulated individually, bromodichloromethane and chloroform are regulated as two of the trihalomethanes included in the MCL for total trihalomethanes. Section 4.2.8 further discusses DBPs.

Table 4.7 summarizes the IOCs detected in the finished water, all of which fall below their MCLs.

Table 4.7 Detected IOCs in the Finished Water

Finished Water Contaminant(1)	Unit	Finished Water MCL	No. of Detects	No. of Samples	Value Range(2)	Average
Ammonia (as N)	mg/L	None	9	36	0.01 – 0.09	0.03
Antimony	mg/L	0.006	1	38	0.00002	0.00002
Barium	mg/L	2	38	38	0.003 – 0.0065	0.004
Chromium	mg/L	0.1	4	38	0.00005 – 0.001	0.0003
Nickel	mg/L	None	1	38	0.0002	0.0002
Nitrate	mg/L as N	10	34	36	0.02 – 0.71	0.29
Nitrate + Nitrite	mg/L as N	10	24	25	0.02 – 0.69	0.29
Selenium	mg/L	0.05	2	38	0.00009 – 0.0002	0.0001
Sodium	mg/L	None	38	38	7.9 – 14.6	9.9

Notes:
Abbreviation: N – nitrogen.
(1) All sampling results from routine grab sampling of the finished water between 2016 and 2021.
(2) Value range is minimum to maximum.

4.2.6 Microbial Contaminants

During compliance sampling that took place between April 2015 and March 2017, JWC sampled for microbial contaminants to comply with the EPA's LT2ESWTR. Table 4.8 summarizes the results for the microbial contaminants sampled.

Table 4.8 Summary of Raw Water Microbial Detections

Raw Water Contaminant	Unit	No. of Detects	No. of Samples	Value Range(2)	Average
E. Coli – Counts(1)	MPN/100 mL	24	24	16 – 816	160
Cryptosporidium(1)	Oocysts/L	4	24	0.1 – 0.3	0.1
Giardia(3)	Cysts/L	10	15	0.1 – 1.5	0.4

Notes:
Abbreviations: MPN – most probable number; mL – milliliter; L – liter.
(1) From LT2ESWTR data submitted to OHA between April 2015 and March 2017.
(2) Value range is minimum to maximum.
(3) From JWC grab sampling data collected as part of LT2ESWTR sampling between January 2016 and March 2017.

E. coli was detected in all samples, and Giardia was detected in two-thirds of the samples. LT2ESWTR sampling for Cryptosporidium is used to classify utilities into bins that determine if additional treatment for Cryptosporidium is required. Cryptosporidium was detected in 17 percent of samples at low concentrations, placing the JWC WTP in the lowest classification, Bin 1, with no additional treatment required.

All three of these microbial contaminants serve as indicators of human or animal waste contamination of the raw water. Agricultural lands in the upstream Tualatin River watershed, combined with human activity, likely contribute to the frequent detection of E. coli and limited, low-level detections of Giardia and Cryptosporidium.

4.2.7 Cyanotoxins and Algal Indicators

At the end of 2018, the OHA established a permanent cyanotoxin monitoring rule for total microcystins and cylindrospermopsin. JWC is currently required to take biweekly samples of the raw water at the intake from May through October, with additional sampling requirements if and when algal toxins are detected.

As part their algal bloom monitoring efforts, JWC samples 2-Methylisoborneol (MIB) and geosmin, which are contributors to taste and odors and indicators of algal presence, in raw water on a monthly basis from March to October and in their finished water as needed, according to detected levels in the raw water and watershed sampling. To date, neither cyanotoxin has been detected in regulatory sampling under the OHA's permanent cyanotoxin monitoring rule.

Table 4.9 summarizes water quality data for MIB and geosmin in JWC's raw and finished water.

Table 4.9 Raw and Finished Water MIB/Geosmin Detections

Contaminant and Sampling Locations	Unit	No. of Detects	No. of Samples	Value Range(3)	Average
MIB – Raw Water(1)	ng/L	5	33	1.0 – 2.6	1.4
MIB – Finished Water(2)	ng/L	4	26	1.0	1.0
Geosmin – Raw Water(1)	ng/L	30	33	1.0 – 8.3	3.2
Geosmin – Finished Water(2)	ng/L	23	26	1.0 – 6.9	3.1

Notes:
Abbreviation: ng/L – nanograms per liter.
(1) Periodic grab sampling for MIB/geosmin in 2016 and 2017. Grab samples collected for MIB and geosmin monthly between March and September beginning in 2018.
(2) Periodic grab sampling in 2016 through 2019. Grab samples collected monthly between March and September beginning in 2020.
(3) Value range is minimum to maximum.

MIB was rarely detected in the raw and finished water and, when it was, at low concentrations. Contrary to MIB, geosmin was frequently detected in both raw and finished water at comparable concentrations of approximately 3 nanograms per liter (ng/L). The similarity in geosmin concentrations in both raw and finished water indicates that little to no removal is being achieved through the JWC WTP.

4.2.8 Distribution System

While sampling in water distribution systems is managed separately by the JWC partners, JWC has regulatory requirements related to the DBP rules and the Lead and Copper Rule. Distribution system water quality is also largely influenced by the finished water quality of each partner's supply sources. As one such water source, the JWC WTP's finished water quality greatly affects the water quality within the partners' distribution systems. This section reviews key parameters for the JWC WTP's finished water and the JWC partners' distribution systems related to the DBP rules and Lead and Copper Rule. Appendix J presents full sampling data for each partner's water quality parameters.

4.2.8.1 Disinfection Byproduct Rules

JWC partners are required to take quarterly samples for two DBPs—total trihalomethanes (bromodichloromethane, dibromochloromethane, bromoform, and chloroform [TTHMs]) and a group of five haloacetic acids (monochloroacetic acid, dichloroacetic acid, trichloroacetic acid, monobromoacetic acid, and dibromoacetic acid [HAA5])—at multiple locations within their distribution systems. As part of OHA-required sampling, JWC must also sample for these DBPs in the WTP's finished water.

Table 4.10 and Table 4.11 summarize TTHM and HAA5 sampling data, respectively, from the JWC WTP's finished water and the partners' distribution systems. For both DBPs, the MCLs shown are based on locational running annual average (LRAAs). The LRAA represents the average of four consecutive quarterly samples at a single location in the distribution system and must remain below the MCL. Both tables show each partner's range and average of all quarterly samples as well as the maximum LRAA, which represents the worst-case location in a distribution system for DBP formation.

Table 4.10 Distribution System TTHMs

Partner	MCL (µg/L)	Range (µg/L)(1)	Average of All Samples (µg/L)	Maximum LRAA (µg/L)
JWC WTP(2)	80	6 – 21	13	N/A
Beaverton(3)	80	20 – 61	33	42
Forest Grove(3)	80	10 – 35	19	24
Hillsboro(3)	80	6 – 60	28	50
TVWD(4)	80	2 – 63	32	40

Notes:
Abbreviations: µg/L – micrograms per liter; N/A – not applicable.
(1) Value range is minimum to maximum.
(2) From data reported to OHA between 2015 and 2021.
(3) From data provided by JWC partners between 2016 and 2021.
(4) Data for 2016 to 2021 from OHA Drinking Water Data Online.

Table 4.11 Distribution System HAA5

Partner	MCL (µg/L)	Range (µg/L)(1)	Average of All Samples (µg/L)	Maximum LRAA (µg/L)
JWC WTP(2)	60	9 – 23	14	N/A
Beaverton(3)	60	3 – 48	24	33
Forest Grove(3)	60	12 – 40	25	31
Hillsboro(3)	60	14 – 45	23	34
TVWD(4)	60	1 – 48	25	32

Notes:
(1) Value range is minimum to maximum.
(2) From data reported to OHA between 2015 and 2021.
(3) From data provided by JWC partners between 2016 and 2021.
(4) Data for 2016 to 2021 from OHA Drinking Water Data Online.

As shown in Tables 4.10 and 4.11, concentrations of both DBPs are low leaving the JWC WTP, at less than 25 percent of the MCL. The low organic content in the raw and finished water and low residence time through the treatment process helps to limit the potential for DBP formation at the WTP, even with low levels of pre-chlorination.

For the partners, average DBP concentrations were less than 50 percent of the MCL with no individual samples exceeding the MCL. In terms of regulatory compliance, maximum LRAAs were well below the MCLs. These results collectively show that neither the JWC WTP's existing treatment processes nor the JWC partners' distribution systems have apparent concerns regarding DBP formation with current pre-chlorination and disinfection practices. Note that DBP MCL exceedances have occurred in the past when pre-chlorination levels were higher and there was lesser oversight of Wapato Lake operations.

4.2.8.2 Lead and Copper Rule

Both lead and copper are also sampled in the JWC WTP's finished water, but compliance with the EPA's Lead and Copper Rule is managed by the JWC partners for their distribution systems. The JWC WTP is also required to maintain a minimum finished water pH of 7.2 for corrosion control related to the Lead and Copper Rule. Compliance with this finished water pH target is discussed in Section 4.2.3.

Regulatory compliance for the Lead and Copper Rule is determined using the 90th percentile concentrations of all lead or copper samples collected during a required sampling period; these concentrations may not exceed their respective established action levels. Utilities are typically required to sample every three years but can be placed under more frequent sampling requirements if the action level is exceeded.

Table 4.12 summarizes the sampling results for lead and copper in the JWC WTP's finished water. Meanwhile, Table 4.13 and Table 4.14 summarize lead and copper sampling, respectively, for each JWC partner, showing the range and average of all sample data as well as the 90th percentile value reported to the OHA during the last three reporting periods.

Table 4.12 JWC WTP Finished Water Lead and Copper Sampling

Contaminant(1)	Action Level (mg/L)	No. of Detects	No. of Samples	Range (mg/L)(2)	Average (mg/L)
Lead	0.015	0	38	0	0
Copper	1.3	3	38	0.0004 – 0.0008	0.0006

Notes:
(1) From routine grab sampling of finished water between 2016 and 2021.
(2) Value range is minimum to maximum.

Table 4.13 JWC Partner Lead Sampling as Part of Lead and Copper Rule Compliance

Partner	Action Level (mg/L)	Range (mg/L)(1, 2)	Average (mg/L)(1)	90th Percentiles for Last Three Reporting Periods (mg/L)(3)
Beaverton	0.015	0.002 – 0.025	0.004	0.003 / 0 / 0.007
Forest Grove	0.015	0.001 – 0.0072	0.0016	0.002 / 0.004 / 0.004
Hillsboro	0.015	0.001 – 0.0126	0.002	0.001 / 0.001 / 0.003
TVWD	0.015	0.00007 – 0.648	0.004	0.011 / 0.012 / 0.010

Notes:
(1) From distribution water quality data provided by the JWC partners between 2016 and 2021.
(2) Value range is minimum to maximum.
(3) Values taken from OHA Drinking Water Data Online.

Table 4.14 JWC Partner Copper Sampling as Part of Lead and Copper Rule Compliance

Partner	Action Level (mg/L)	Range (mg/L)(1, 2)	Average (mg/L)(1)	90th Percentiles for Last Three Reporting Periods (mg/L)(3)
Beaverton	1.3	0.051 – 0.437	0.099	0.127 / 0.169 / 0.252
Forest Grove	1.3	0.007 – 0.634	0.139	0.269 / 0.435 / 0.421
Hillsboro	1.3	0.00624 – 0.264	0.038	0.065 / 0.073 / 0.110
TVWD	1.3	0.002 – 0.941	0.130	0.249 / 0.199 / 0.274

Notes:
(1) From distribution water quality data provided by the JWC partners between 2016 and 2021.
(2) Value range is minimum to maximum.
(3) Values taken from OHA Drinking Water Data Online.

Lead and copper are rarely detected in the finished water leaving the JWC WTP, indicating that corrosion in premise plumbing is likely the main contributor to the lead and copper found in the partners’ distribution systems. Lead concentrations in individual tap samples collected from Beaverton and TVWD’s systems have exceeded the action level. Note that TVWD receives water from the Portland Water Bureau (PWB) in addition to JWC. TVWD exceeded the action level for lead in 2013 and 2016 under PWB’s Joint Monitoring Program for consecutive systems, but the 90th percentile in the last three periods of compliance sampling has remained below the action level under TVWD’s stand-alone monitoring program which began in 2017.

In anticipation of the Willamette Water Supply System (WWSS) being integrated into three of the partners’ supplies in the future, JWC is evaluating further corrosion control efforts related to finished water pH and alkalinity targets. The initial evaluation, provided in Appendix G, determined that increasing the finished water pH target to be more compatible with the WWSS water quality may be beneficial for reducing lead and copper solubility. Further investigation into potential DBP formation is recommended prior to JWC setting a new finished water pH target for corrosion control.

4.3 Contaminants of Emerging Concern and Future Regulations

This section discusses CECs with a focus primarily on the EPA's Unregulated Contaminant Monitoring Program (UCM), which encompasses contaminants that are most likely to be subject to future regulation.

4.3.1 Unregulated Contaminant Monitoring

As specified in the Unregulated Contaminants Monitoring Rule (UCMR), the EPA manages and uses the UCMR to collect data on the occurrence and concentrations of contaminants that may be in drinking water but are not currently regulated by the Safe Drinking Water Act (SDWA). Through nationwide monitoring, this program helps to understand how widespread the occurrence of a particular contaminant is, the approximate population exposed to this contaminant, and the level of that exposure.

Every five years, the EPA issues a list of contaminants for sampling and monitoring under the UCMR, with the majority of selected contaminants taken from the Contaminant Candidate List. The SWDA Amendments of 1996 established the following for the UCMR:

- Monitor no more than 30 contaminants every five years.
- Monitor only a representative sample of public water systems serving fewer than 10,000 people (all public water systems serving more than 10,000 people are required to monitor for the selected contaminants).
- Store analytical results in a National Contaminant Occurrence Database (NCOD).

Thus far, the UCMR has progressed in rounds, the most recent of which, round 4 (UCMR4), was completed by utilities from 2018 through 2020. The proposal for round 5 (UCMR5) was published on March 11, 2021, with sampling to take place from 2023 through 2025.

The following sections discuss results from JWC partners' participation in the most recent two UCMR rounds, UCMR3 and UCMR4, and expectations for UCMR5.

4.3.1.1 UCMR3 Summary

JWC partners completed UCMR3 sampling between 2013 and 2015, the results of which are summarized in Table 4.15. Table 4.15 shows contaminants that were detected at entry points (i.e., interconnections) to the partners' distribution systems for the JWC WTP; note that entry point sampling for partners' other supplies is not shown. Table 4.15 includes the EPA's contaminant reference concentrations, which are health-based values that provide context for the detection of a contaminant, and the range of detections observed throughout Oregon and Washington to provide context for regional contaminant detections.

Table 4.15 Summary of Detected UCMR 3 Results for JWC WTP Water at JWC Partners Entry Point to Distribution System

Contaminant	Reference Concentration	Oregon / Washington Detection Range (µg/L)	JWC Partners Range (µg/L)(1)
Chromium	100	0.2 – 55	0.20 – 0.34
Chromium-6	N/A	0.03 – 9.9	0.098 – 0.232
Strontium	1,500	0.9 – 620	29.0 – 87.7
Vanadium	21	0.2 – 50	0.25 – 0.77

Note:
(1) Data presented is for JWC WTP water entry point sampling at partner interconnections only. Other UCMR3 sampling conducted by JWC partners is not shown.

Four contaminants were detected in JWC's finished water entering the partners' distribution systems, but detected concentrations were well below their reference concentrations. All detected contaminants are naturally occurring metals in the Pacific Northwest.

All other UCMR3 contaminants that were tested but not detected, including six perfluorinated substances, are included in Appendix J.

4.3.1.2 UCMR4 Summary

JWC partners completed UCMR4 sampling in 2018 and 2019, the results of which are summarized in Table 4.16. As with Table 4.15, Table 4.16 presents detected contaminants from sampling conducted at entry points from the JWC WTP to the partners' distribution systems and as well as results within those systems for select DBPs included in UCMR 4 sampling. Table 4.16 also shows the EPA's reference concentrations and a range of detections observed throughout Oregon and Washington.

Table 4.16 Summary of Detected UCMR 4 Results for JWC Partners

Contaminant	Sampling Location	Reference Concentration ((µg/L)	Oregon / Washington Detection Range (µg/L)	JWC Partners Range (µg/L)
Manganese	Entry Point⁽¹⁾	300	0.4 – 1,590	0.4 – 19.0
HAA5	Distribution System⁽²⁾	60	0.2 – 130.8	8.7 – 41.4
HAA6Br	Distribution System⁽²⁾	N/A	0.3 – 26.48	0.30 – 5.28
HAA9	Distribution System⁽²⁾	N/A	0.2 – 131.32	0.01 – 43.72

Notes:

Abbreviations: HAA6Br – a group of six brominated haloacetic acids (bromochloroacetic acid, bromodichloroacetic acid, dibromoacetic acid, dibromochloroacetic acid, monobromoacetic acid, and tribromoacetic acid);
HAA9 – a group of nine haloacetic acids (bromochloroacetic acid, bromodichloroacetic acid, chlorodibromoacetic acid, dibromoacetic acid, dichloroacetic acid, monobromoacetic acid, monochloroacetic acid, tribromoacetic acid, trichloroacetic acid).

(1) From JWC partner sampling point at the entry point to the distribution system for JWC interties.

(2) From JWC partner sampling at DBP compliance sites within partners' distribution systems.

Manganese was detected in the JWC WTP's finished water at concentrations well below the reference concentration or secondary MCL. Low-level detects of manganese are consistent with results from regular sampling of the WTP's finished water.

Three DBPs were detected in the partners' distribution systems. Consistent with results from finished water and distribution system sampling, these DBP concentrations were low, and the detected HAA5 concentrations were well below the MCL.

All other UCMR4 contaminants that were tested but not detected, including multiple algal toxins, are included in Appendix J.

4.3.1.3 UCMR5 Sampling

UCMR5 sampling is anticipated to take place between 2023 and 2025. Published in March 2021, the proposal for UCMR5 includes monitoring for 29 per- and polyfluoroalkyl substances (PFAS) and lithium. UCMR3 included sampling for six PFAS that are proposed for additional sampling in UCMR5. Though these six PFAS were not detected in the JWC WTP's finished water entering the partners' distribution systems during UCMR3, the minimum detection levels for sampling are an order of magnitude lower for the upcoming UCMR5.

4.3.2 Future Regulations

The JWC WTP's *2018 Facility Plan* indicated that future regulations most likely to affect JWC are additional or more stringent DBP regulations and the regulation of CECs related to pesticides and herbicides. This section reviews other potential future regulations, as well as revisions to existing regulations, that may influence JWC and their partners.

4.3.2.1 Perfluorinated Substances

In February 2020, the EPA issued a determination to regulate two perfluorinated substances, perfluorooctanoic acid (PFOA) and perfluorooctanesulfonic acid (PFOS). As discussed in Section 4.3.1, none of the JWC partners detected these substances during UCMR3. While detections may be possible under the UCMR5's lowered detection levels, regulations placed on these two substances are not anticipated to require additional treatment for regulatory compliance.

High levels of these perfluorinated substances are most often tied to point sources of contamination, such as air force bases or fire-fighting training facilities. Without any known sources affecting JWC's raw water supply, these contaminants are unlikely to be significant concerns for JWC.

4.3.2.2 Disinfection Byproducts

The EPA is currently reviewing the Microbial and DBP Rules as part of their most recent Six-Year Review, which identifies the potential need for revisions to such rules. The following potential revisions are under consideration:

- Establish a numeric limit on minimum distribution system residual to reduce Legionella occurrences.
- Change the DBP rules to include additional brominated DBPs that are more toxic than currently regulated TTHMs and HAA5.
- Further review of chlorate and N-Nitrosodimethylamine (NDMA).

Changes to the Microbial and DBP Rules are not anticipated to significantly affect JWC. While maintaining a minimum distribution system chlorine residual may potentially require modifications to the target finished water leaving the JWC WTP, monitoring within distribution systems will be the responsibility of the individual partners. Brominated DBPs were included in UCMR4 sampling, and DBPs leaving the JWC WTP and within the partner systems were well below current MCLs. Additionally, NDMA is a byproduct of disinfection with chloramines, which are not used at the JWC WTP. Finally, Chlorate, formed from the degradation of sodium hypochlorite or as a byproduct of disinfection with chlorine dioxide, is discussed further in Section 4.3.2.5.

4.3.2.3 Lead and Copper Rule Revisions

To strengthen public health protections and clarify the Lead and Copper Rule's implementation requirements, the EPA released proposed Lead and Copper Rule Revisions (LCRR) on October 10, 2019. The proposal includes a new trigger level for lead at 10 parts per billion (ppb). Utilities exceeding this level will be required to re-optimize their existing treatment for corrosion control.

The EPA recently extended the effective date of the LCRR to December 16, 2021, and the compliance date to October 16, 2024. These delays are intended to provide for further review of the rule which could result in modifications or even full withdrawal of the published rule.

Lead and copper have generally not been detected in the JWC WTP's finished water, and JWC partners are currently responsible for their distribution systems' compliance with the Lead and Copper Rule. As noted in Section 4.2.8, TVWD had 90th percentile lead levels exceed the action level in 2013 and 2016 under the PWB's Joint Monitoring Program, but detections were below the action level in the three most recent sampling events under TVWD's individual monitoring program. For the last three reporting periods, TVWD has been the only partner with 90th percentile lead levels that exceed the proposed level of 10 ppb.

Additional discussion of the Lead and Copper Rule Revisions and the implications for the JWC partners is provided in Appendix G. As mentioned, Appendix G also presents the results of the initial water quality blending evaluation in anticipation of the WWSS being integrated into three partners' water supplies in the future.

4.3.2.4 Manganese

In 2019, Canada established two new values regulating manganese in drinking water: A health-based maximum allowable concentration (analogous to the U.S's primary contaminant MCL) of 0.12 mg/L and an aesthetic objective (analogous to the U.S.'s secondary contaminant MCL) of 0.02 mg/L. The aesthetic objective for manganese is below the EPA's current secondary MCL of 0.05 mg/L.

Manganese was included in sampling for UCMR4 to help assess if it should be moved from a secondary MCL to a primary MCL. As of March 10, 2020, the EPA determined there was not enough information to proceed to the regulatory determination assessment phase.

However, the EPA has established health advisory levels (HALs) for manganese. HALs establish a specific duration and threshold concentration of exposure below which no health effects are anticipated to occur. For all persons, EPA has a 1-day and 10-day HAL of 1 mg/L and a lifetime HAL of 0.3 mg/L. For bottle-fed infants younger than six months, the EPA also has a 10-day HAL health advisory level of 0.3 mg/L.

As shown in Section 4.2.4, manganese concentrations in the JWC WTP's finished water have been below both levels established in Canada, indicating that the plant's current practice of pre-chlorination and subsequent oxidation of manganese has been effective. If pre-chlorination continues, no additional treatment is anticipated to be required should manganese be converted to a primary MCL.

4.3.2.5 Chlorate

As part of their Six-Year Review of the Microbial and DBP Rules, the EPA is currently considering the regulation of chlorate. Although it was also sampled as part of UCMR3, chlorate has not yet reached the regulatory determination assessment phase. As the Six-Year Review progresses, further steps towards regulation may occur, but potential MCL values or likelihood of regulation are unclear at this time.

Chlorate was not detected in the JWC WTP's finished water during UCMR3 since chlorine gas is currently used for disinfection. If JWC were to switch to on-site sodium hypochlorite generation or bulk sodium hypochlorite, chlorate will be present in the finished water.

No treatment process effectively removes chlorate. JWC can manage chlorate levels by controlling its formation through operational strategies such as managing sodium hypochlorite storage times, establishing purity levels for suppliers, and controlling the dilution levels of sodium hypochlorite. If JWC were to transition to using sodium hypochlorite in the future, chlorate levels are not anticipated to be a concern, provided that due diligence is taken regarding the operational strategies listed above.

4.4 Water Quality Challenges and Process Capabilities

To identify potential treatment process gaps that may form the basis for future improvements, potential water quality challenges that may affect the JWC WTP were identified. More specifically, the following water quality constituents were identified as having the potential to pose water quality challenges:

- Regulated Pathogens.
- High turbidity.
- Algal toxins.
- Tastes and Odors.
- High TOC and DBPs (TTHMs, haloacetic acids, bromate, chlorate).
- Metals.
- Pesticides.

Table 4.17 presents existing treatment processes at the JWC WTP and resulting level of treatment provided for each water quality constituent of concern.

Table 4.17 Treatment for Water Quality Constituents of Concern with Existing WTP Processes

Water Quality Constituent	Pre-chlorination	PAC	Flocculation / Sedimentation	Anthracite Filters	Chlorine Disinfection
Pathogens	Partial Treatment	-	-	Full Treatment	Full Treatment
High Turbidity	-	-	Full Treatment	Partial Treatment	-
Algal Toxins	Limited (cell lysis can release toxins)	Partial Treatment (when in use)	-	-	-
Tastes and Odors	-	Full Treatment (when in use)	-	-	-
High TOC and DBPs (Trihalomethanes, Haloacetic acids, bromate, chlorate)	Limited (contributes to DBP formation)	Full Treatment (when in use)	Partial Treatment	Partial Treatment	-
Metals	Partial Treatment	-	-	Partial Treatment	-
Pesticides	-	Partial Treatment (when in use)	-	-	-

Note:
Abbreviation: PAC - powdered activated carbon.

The JWC WTP provides multiple continuous barriers and treatment processes for pathogens and turbidity. Only partial treatment is provided, or full treatment is only provided when powdered activated carbon (PAC) is online for most of the water quality constituents. Note that many of the water quality constituents treated with PAC are either not routinely present in the raw water or are at very low levels of concern.

In addition to the current treatment processes at the WTP, JWC maintains an active source water protection program. This program allows to identify and mitigate potential source water quality risks prior to the WTP. Robust watershed protection reduces the likelihood of the JWC WTP experiencing many of the water quality challenges discussed in this section.

While PAC is a versatile and effective treatment process for adsorbing a wide variety of contaminants, it is shown only to offer partial treatment since its performance depends on several factors. For one, the water quality constituents shown in Table 4.17 encompass a broad spectrum of contaminants, and PAC is more effective at removing certain contaminants than others. This condition was observed in 2019 when JWC conducted benchtop testing to determine how effective two different types of PAC types are in removing four algal toxins. Results showed that both PAC types preferentially removed certain algal toxins more than others.

Along these lines, treatment performance can also be influenced by the type of PAC used. PAC can be manufactured from wood, coconut shells, coal, or other materials. Different PAC formulations may be more effective at removing a certain contaminant type than others.

Finally, PAC effectiveness depends on the dose and contact time (CT). Different doses may be required depending on the contaminant being targeted, and sufficient CT must be provided to ensure effective removal.

A major challenge in employing PAC to address water quality constituents of concern is its effect on other treatment processes. PAC must be settled out and then removed from the process stream and sent to solids handling, which greatly increases the volume of solids that must be processed through solids handling processes and potentially increases settled water turbidity.

Currently, the JWC WTP's solids drying beds are filled for three to six months before being rotated out of service to dry the solids before they are hauled away by truck. Extended use of PAC would reduce each solids drying bed's filling time, requiring more frequent rotation between the drying beds as well as increased truck hauling. Depending on duration and concentration, extended PAC use could also reduce the beds' drying effectiveness, which will require JWC to haul a greater volume of wetter solids to maintain drying bed capacity.

PAC use also affects pre-chlorination strategies. Since PAC adsorbs chlorine, pre-chlorination at rapid mix should be discontinued when PAC is used. During these periods, the chlorine dose added to the settled water that enters the filters may need to be adjusted to ensure that a free chlorine residual is maintained through the filters. Provided that an oxide coating has already been formed on the filter media, chlorination at the settled water channel should be sufficient to ensure continued manganese oxidation and capture on the filter media.

JWC currently has two PAC towers installed, with a third unit purchased but not yet incorporated into the treatment process as of July 2021. Table 4.18 summarizes the JWC WTP's current PAC feed capacity at a range of plant demands and the anticipated PAC feed capacity with the third feeder online.

Table 4.18 PAC Feed Capacity

PAC Dose (mg/L)	Maximum Plant Production – 2 PAC Towers (mgd)	Maximum Plant Production – 3 PAC Towers
55	30	45
37	45	60
30	55	70
25	65	80
22	75	90
19	85	100

As discussed, PAC effectiveness can be highly dependent on dose. With the two existing feeders online, the PAC dose is limited to around 20 mg/L at peak summer production rates. During the previous benchtop testing in 2019, PAC doses of at least 20 mg/L were generally needed to provide at least 50 percent removal of the tested algal toxins. PAC doses of 50 mg/L were needed to provide greater than 80 percent removal of the tested algal toxins. Even with the addition of a third PAC feeder, the plant's maximum production may be limited if doses higher than 25 mg/L are required.

While PAC provides JWC with a treatment barrier for many water quality challenges of concern, it remains an inherently reactive treatment barrier. That is, PAC is typically dosed in response to a specific water quality event and is not typically intended to serve as a consistent or proactive control measure for potential water quality concerns. As such, continued PAC use is not recommended unless significant changes are made to the WTP's operations and solids handling processes. Instead, JWC may consider a more proactive, continuous treatment barrier to manage water quality challenges of concern.

4.5 Water Quality Goals and Recommendations

JWC's primary water quality goal is to supply water that meets or exceeds all regulatory requirements. Table 4.19 summarizes JWC's additional water quality goals and operational targets to guide the operation of the WTP. These goals include existing JWC goals, industry trends, goals for other water supply sources, and are consistent with JWC's primary mission to provide excellent drinking water.

Several of these goals go beyond regulatory compliance or set targets for currently unregulated parameters. Achieving these goals indicates the plant is performing to a high level. Though infringing on these goals does not necessarily indicate an immediate treatment issue, chronic infringement may be a trigger for operational or infrastructure modifications.

Table 4.19 JWC Water Quality Goals

Parameter	Regulatory Requirement	Goal	Source
Combined Filter Effluent Turbidity	< 0.3 NTU 95 percent of time. Never to exceed 1.0 NTU.	<0.1 NTU	2020 Operations Manual
Free Chlorine	0.2 mg/L – 4.0 mg/L	1.10 mg/L	2020 Operations Manual
pH	>7.2 Daily Average	7.7⁽⁴⁾	2020 Operations Manual
TOC	N/A(1)	35 – 45 percent of raw water TOC(2)	2020 Operations Manual
Virus Inactivation	4.0 log	>4.0 log	2020 Operations Manual
Giardia Inactivation	3.0 log	>3.0 log	2020 Operations Manual
Cryptosporidium Inactivation	3.0 log	>3.0 log	2020 Operations Manual
Iron	0.3 mg/L(3)	<0.1 mg/L	*Iron and Manganese from JWC Supply TM* (Jacobs, 2018)
Manganese	0.05 mg/L(3)	<0.02 mg/L	*Iron and Manganese from JWC Supply TM* (Jacobs, 2018)
Aesthetics	3 TON(3)	Provide aesthetically pleasing water, in both appearance and taste and odor. No objectionable taste and odor year-round.	2020 Operations Manual Adapted from goals established for WWSS.
TTHM	80 µg/L	40 µg/L (50 percent of MCL)	Adapted from goals established for WWSS.
HAA5	60 µg/L	30 µg/L (50 percent of MCL)	Adapted from goals established for WWSS.
Geosmin	N/A	< 7 ng/L	Adapted from goals established for WWSS.
SOCs	Varies	No detects for current detection limits.	Adapted from goals established for WWSS.
VOCs	Varies	No detects for current detection limits.	Adapted from goals established for WWSS.
IOCs	Varies	< 50 percent of MCLs for regulated contaminants.	Adapted from goals established for WWSS.
Arsenic	0.01 mg/L	< 2 µg/L	Adapted from goals established for WWSS.
Sulfate	250 mg/L(3)	< 250 mg/L	Adapted from goals established for WWSS.
Contaminants of Emerging Concern	N/A	Position for compliance with future water quality regulations as they are developed. Target <80 percent of MCL on future regulatory limits.	Adapted from goals established for WWSS.
Algal Toxins	Varies	No detects in finished water for currently regulated cyanotoxins, cylindrospermopsin and total microcystins.	Adapted from goals established for WWSS.
Perfluorinated Compounds		No detects in finished water.	Adapted from goals established for WWSS.

Notes:

(1) No regulatory requirement because raw water TOC is less than the 2.0 mg/L level that would trigger removal requirements.

(2) Range of TOC reduction based on existing regulatory removal requirements. 35 percent reduction in TOC is required if raw water TOC is between 2 and 4 mg/L, and 45 percent reduction in TOC is required if raw water TOC is between 4 and 8 mg/L.

(3) Secondary MCL for aesthetic purposes.

(4) Finished water pH target may be adjusted in the future for compatibility with WWSS water quality.

In addition to the water quality goals presented in Table 4.19, JWC has a policy to notify customers on their industrial user list when the water quality parameters shown in Table 4.20 are met or exceeded. The parameters listed have been set by these industrial users, who are sensitive to water quality changes that may affect their business operations but do not constitute a health or regulatory concern. Under normal water quality conditions, JWC operates the WTP to minimize the frequency of exceedances and impacts to these unique users.

Table 4.20 Water Quality Parameters and Trigger Levels for Industrial User Notification

Parameter	Trigger Level for Industrial User Notification
Aluminum Sulfate Dosage	>40 mg/L
Caustic Soda Usage	Commencement and conclusion of dosage at rapid mix.
Chlorine Dosage	>3 mg/L
Coagulant Aid Polymer Usage	Commencement and conclusion of dosage at rapid mix.
Filter Aid Polymer Dosage	>0.2 mg/L
Finished Water pH	Outside target range of 6.8 to 8.7.
Finished Water TOC	>1.0 mg/L
Powdered Activated Carbon Usage	Commencement and conclusion of dosage.
Other Water Quality Irregularities or Major Operational Changes	JWC activities that may impact industrial user business processes and operations.

4.5.1 Additional Recommendations

The following categorized sets of recommendations and considerations strive to improve the JWC WTP's existing treatment capabilities and allow JWC to continue meet all potential future water quality regulations:

- Powdered Activated Carbon Use:
  - Develop operational strategies and determine hauling requirements to ensure the WTP's existing solids handling processes are not overwhelmed should a water quality event necessitate the prolonged use of PAC.
  - Conduct additional benchtop testing of PAC types and dosages, as needed, to address specific water quality challenges of concern.
  - Consider a more proactive treatment approach, such as ozone, to address water quality challenges if PAC use becomes frequent. PAC is a reactive treatment approach whose prolonged use may have negative effects on the WTP's downstream and solids handling processes.
- Source Water Protection:
  - Continue to develop harmful algal bloom forecasting tools with the Water Research Foundation project. In addition, consider leveraging and expanding these efforts to develop hydrodynamic watershed modeling that will allow JWC to predict and quantify water quality impacts of short-term weather events or land use changes in the watershed.
- Disinfectant Source:
  - Secure a high-quality supply of sodium hypochlorite to manage chlorate/chlorite formation should JWC decide to convert from chlorine gas to bulk sodium hypochlorite for disinfection.

Chapter 5

WATER DEMAND FORECAST AND SUPPLY STRATEGY

5.1 Introduction

This chapter presents Joint Water Commission's (JWC's) historical water production and partner supply allocations, existing water rights, and water demand forecast. The analysis found that the JWC Water Treatment Plant (WTP) is expected to have sufficient capacity to meet projected customer demands over the 20-year planning horizon of this master plan (years 2023-2042). While individual partners may exceed their current allocations during that time, they are expected to be able to meet that demand through alternative water supply sources or by leasing excess capacity from other partners. However, demand projections should be re-evaluated with each master plan update in the future to ensure the continued ability for the JWC to meet partner demands.

5.2 Historical Water Production

On average, the JWC produces and distributes approximately 35,000 acre-feet per year (AFY) of treated water to their partners, the Cities of Hillsboro, Beaverton, and Forest Grove and Tualatin Valley Water District (TVWD). The most recent historical peak production occurred in 2015 when JWC produced and distributed 38,600 acre-feet (ac-ft) of water.

As described in Chapter 2, the JWC WTP now has a rated capacity of 85 million gallons per day (mgd) following a 10-mgd plant expansion that occurred in 2019. As shown in Figure 5.1, demand on the plant fluctuates throughout the year, with daily peak production below 30 mgd during the winter months and above 50 mgd in the summer months. Production reached its historical peak in July 2015 with a maximum daily production of 65 mgd.

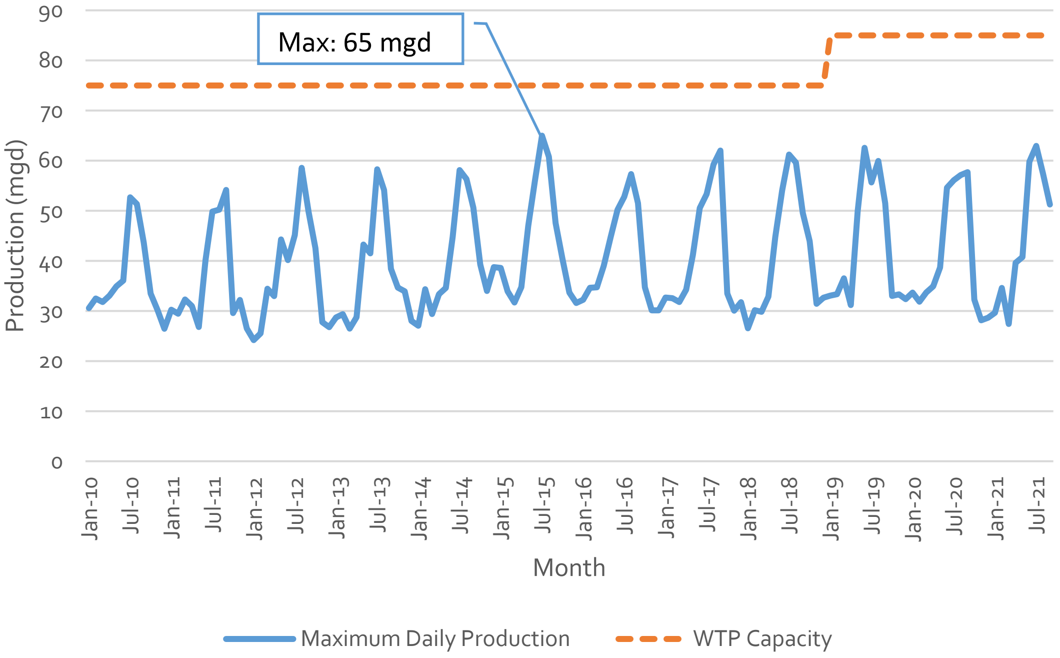


Figure 5.1 Maximum Daily JWC WTP Production by Month

Table 5.1 shows the WTP's historical total annual production and maximum daily production.

Table 5.1 Historical Total Annual Production and Maximum Daily Production

Year	Total Annual Production (AFY)	Maximum Daily Production (mgd)
2010	31,300	52.7
2011	30,400	54.2
2012	32,000	58.5
2013	32,000	58.3
2014	35,700	58.1
2015	38,600	65.0
2016	37,000	57.3
2017	36,100	62.0
2018	36,900	61.2
2019	36,100	62.6
2020	34,900	57.7

5.2.1 Partner Demands

As described in Chapter 2 and shown in Table 5.2, each one of JWC's partners owns a percentage of the water produced at the JWC WTP. If a partner's demand exceeds their level of ownership, they can lease additional WTP capacity from the other partners if available. If partners' projected demands from the JWC WTP consistently exceed their allocation such that total plant capacity is not sufficient, individual or multiple partners may initiate a plant expansion.

In addition to providing water to the partners, JWC also sells water to two wholesale customers: the City of North Plains and the Westside Lutheran School. North Plains receives up to 1.2 mgd of water supply from the JWC WTP. North Plains' demands are applied to the partners' allocation and is taken into account when considering total WTP capacity. The Westside Lutheran School is a small connection, serving approximately 230 students and teachers and purchases a small amount of water from JWC.

Table 5.2 JWC WTP Ownership by JWC Partners

JWC Partner	Ownership Percentage	Ownership Flow (mgd)
City of Hillsboro	49.12%	41.75
City of Beaverton	22.06%	18.75
TVWD	17.06%	14.50
City of Forest Grove	11.76%	10.00
Total	**100%**	**85**

The following sections describe the historical demand from the JWC system for each partner including North Plains. Note that most partners also receive water supplies from other sources, so the historical demands in this section do not reflect the entire water demand for each partner.

5.2.1.1 City of Hillsboro

Hillsboro's current allocation from the JWC WTP is 41.75 mgd. Prior to the plant's expansion in 2019, their allocation was 33.75 mgd. As shown in Figure 5.2, Hillsboro's historical maximum daily demand from JWC has been relatively consistent around 30 mgd, peaking at 30.9 mgd in August 2021.

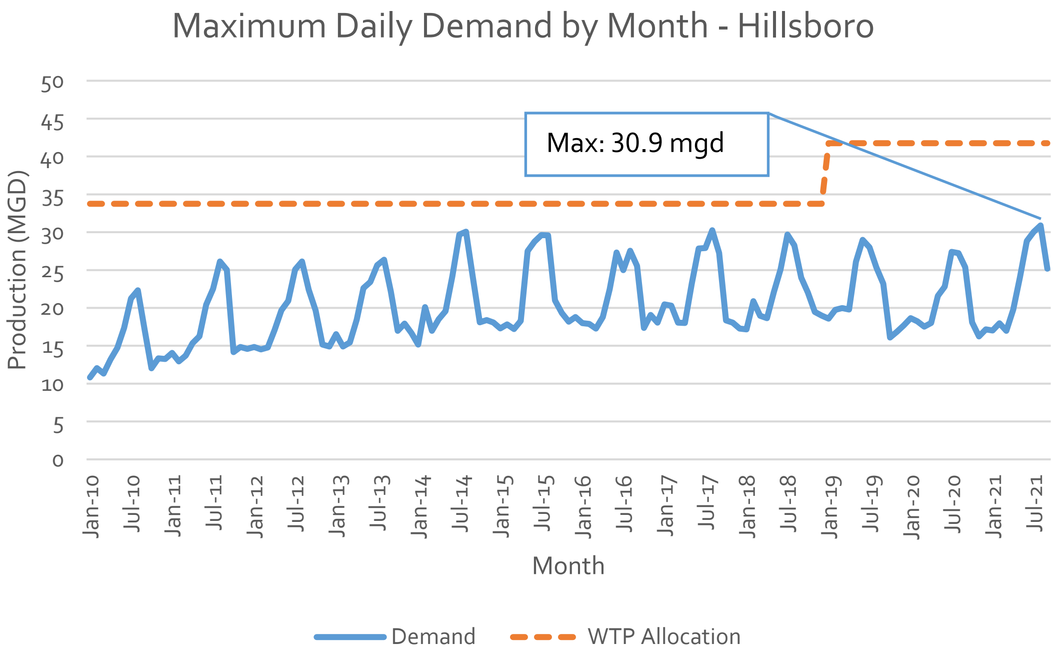


Figure 5.2 Hillsboro Historical Maximum Daily Demand by Month

5.2.1.2 City of Beaverton

Because they did not participate in the 2019 plant expansion, Beaverton's allocation from the JWC WTP has remained constant at 18.75 mgd in recent years. As shown in Figure 5.3, Beaverton's maximum daily demand from JWC has gradually increased from approximately 11 mgd to approximately 14 mgd over the past decade, peaking at 14.3 mgd in July 2020.

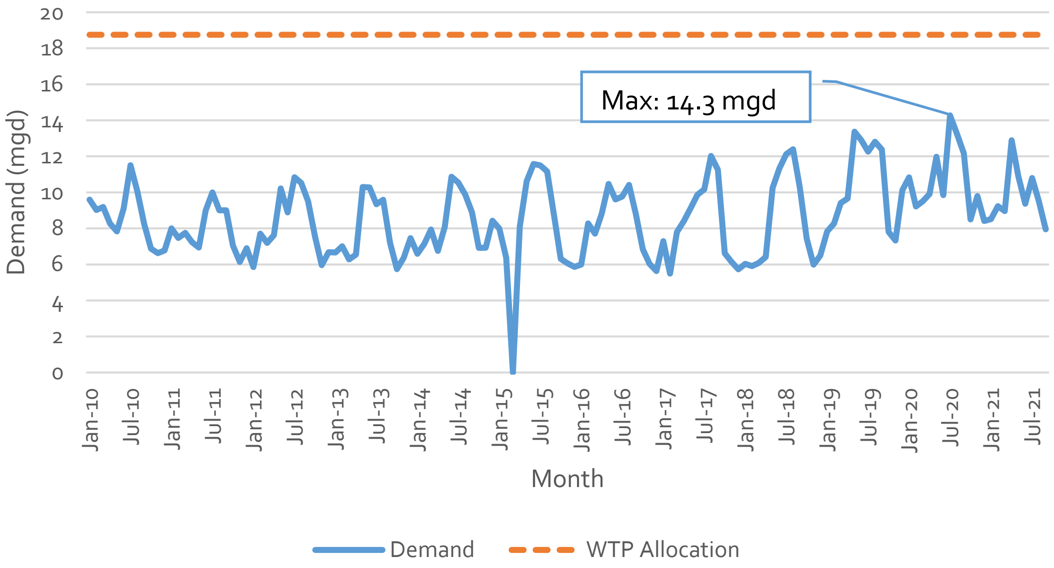


Figure 5.3 Beaverton Historical Maximum Daily Demand by Month

5.2.1.3 TVWD

TVWD's current allocation from the JWC WTP is 14.5 mgd. Prior to the plant expansion in 2019, their allocation was 12.5 mgd. As shown in Figure 5.4, TVWD's maximum daily demand from JWC has historically fluctuated between 12 and 18 mgd, peaking at 17.6 mgd in July 2015. Whenever their demand exceeds their allocation, TVWD has leased additional JWC WTP capacity from other partners.

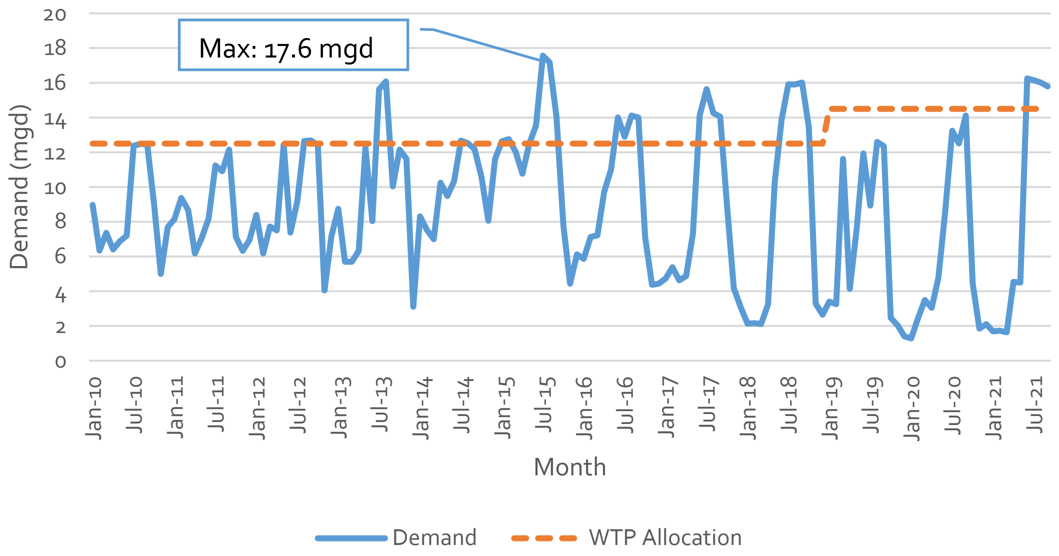


Figure 5.4 TVWD Historical Maximum Daily Demand by Month

5.2.1.4 City of Forest Grove

Forest Grove's current allocation from the JWC WTP is 10 mgd. Like Beaverton, Forest Grove did not participate in the 2019 plant expansion, so their allocation has remained constant in recent years. As shown in Figure 5.5, Forest Grove's maximum daily demand from JWC has stayed relatively consistent around 4 to 5 mgd over the past decade, peaking at 5.2 mgd in July 2015.

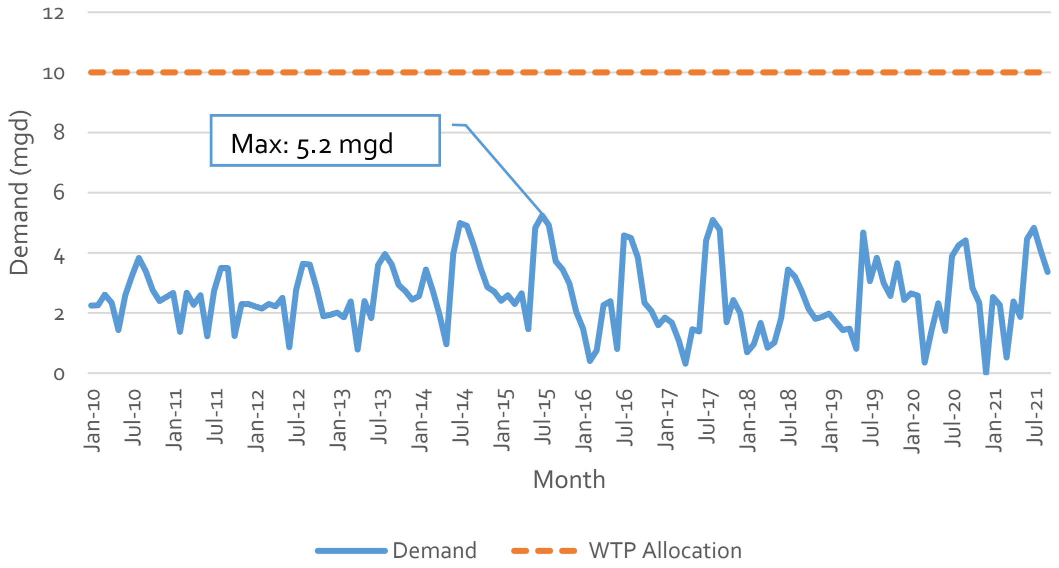


Figure 5.5 Forest Grove Historical Maximum Daily Demand by Month

5.2.1.5 City of North Plains

While not a JWC partner, North Plains has an intergovernmental agreement (IGA) with JWC to purchase up to 1.2 mgd of water from JWC. As shown in Figure 5.6, North Plains' maximum daily demand from JWC has typically been less than 1 mgd but has steadily increased over the past decade.

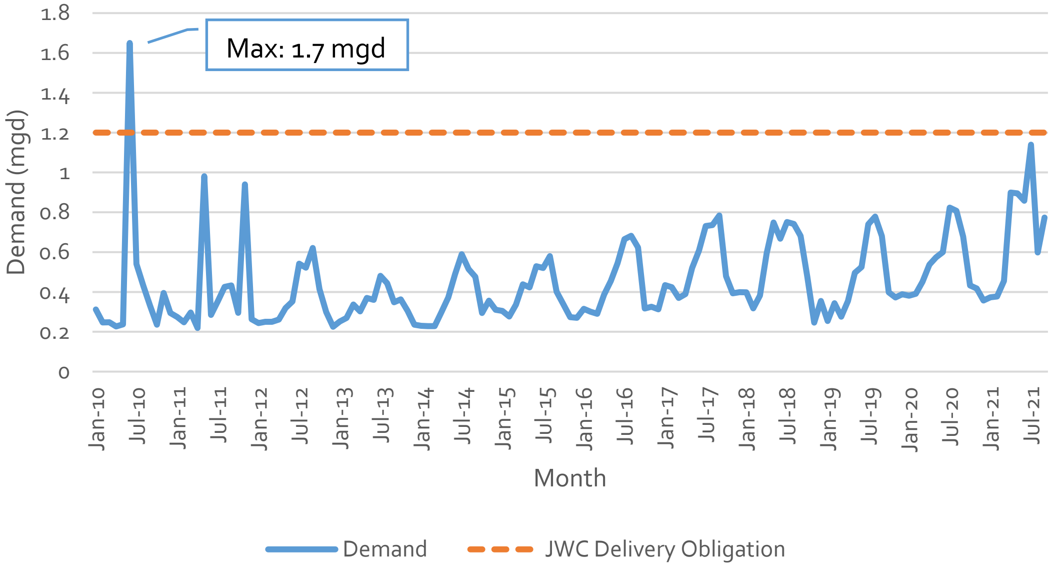


Figure 5.6 North Plains Historical Maximum Daily Demand by Month

5.3 Water Rights and Licenses

JWC currently sources their water supply from the Tualatin River and its tributaries, which include a combination of natural flow and stored water released from Hagg Lake and Barney Reservoir in the Trask River watershed.

JWC's use of these sources is authorized by numerous water rights and licenses, including rights for the use of natural flow, storage rights, secondary rights to use stored water, and a limited license (LL) for aquifer storage and recovery (ASR). These water rights and ASR license authorize the use of water for municipal purposes, with the exception of two rights for pollution abatement. Some water rights are held by the JWC while others are held by the Barney Reservoir Joint Ownership Commission (BRJOC), and still others are held by the individual partner agencies.

Table 5.3 shows all water rights for municipal water use within the JWC system.

Table 5.3 JWC Water Rights

Application	Permit	Certificate	Source	Source Type	Priority Date	Authorized Rate (cfs)	Authorized Rate (mgd)	Authorized Volume (ac-ft)	Entity name on water right	Type of Beneficial Use	Authorized Date of Completion	Notes
S-2016	S-1136	81026	Sain Creek	Natural Flow	1/22/1912	3	1.94	-	City of Hillsboro	Municipal	-	-
S-4250	S-2443	81027	Sain Creek	Natural Flow	5/1/1915	2	1.29	-	City of Hillsboro	Municipal	-	-
S-22251	S-17549	~~85113~~	Gales Creek	Natural Flow	2/14/1947	4.46	2.88	-	City of Forest Grove	Municipal	10/1/2035	Inchoate Transfer T-11677
S-13681	S-10408	67891	Tualatin River	Natural Flow	8/15/1930	9	5.82	-	City of Hillsboro	Municipal	-	-
S-51643	S-46423	85913	Tualatin River	Natural Flow	2/6/1974	43	27.79	-	City of Hillsboro	Municipal	-	-
S-60357	S-45455	85914	Tualatin River	Natural Flow	7/15/1980	25	16.16	-	City of Beaverton	Municipal	-	-
S-54203	S-40615	85916	Tualatin River	Natural Flow	4/28/1976	33	21.33	-	City of Forest Grove	Municipal	-	-
S-88506	-	-	Tualatin River	Natural Flow	1/31/2018	44	28.44	-	Joint Water Commission	Municipal	-	Under review. Proposed season of use is limited to December through April. JWC proposal was a joint rate limitation for Application S-88056 and Permit S-54737.
S-69637	S-54737	-	Scoggins Creek	Natural Flow	6/9/1988	75	48.47	-	City of Hillsboro, City of Forest Grove, City of Beaverton, Tualatin Valley Water District, Joint Water Commission	Municipal	10/1/2071	Season of use limited to October through May.
R-32420	R-4890	81024	Middle Fork of the North Fork Trask River (Barney Reservoir)	Storage	6/26/1958; 12/10/1965	-	-	12600; 7400	City of Hillsboro	Municipal	-	-
S-32421	S-32139	81020	Middle Fork of the North Fork Trask River and Barney Reservoir	Natural Flow and Stored Water Secondary	6/26/1958	38.7	25.01	-	City of Hillsboro	Municipal	-	-
S-48359	S-37837	81022	Barney Reservoir	Stored Water Secondary	6/24/1971	-	-	500	City of Forest Grove	Municipal	-	-
S-88492	S-55219	-	Barney Reservoir	Stored Water Secondary	12/26/2017	30	19.39	8734	Barney Reservoir Joint Ownership Commission	Municipal	-	-
R-38449	R-5777	81149	Scoggins Creek	Storage	2/20/1963	-	-	13500	Bureau of Reclamation	Irrigation, supplemental irrigation, municipal, water quality control, and fish and recreation	-	60,000 ac-ft total, 13,500 ac ft for municipal use by JWC partners.
S-38447	S-35792	87304	Scoggins Reservoir/ Henry Hagg Lake	Stored Water Secondary	2/20/1963	70	45.24	13,000	Bureau of Reclamation	Municipal	-	-
S-38447	S-35792	93873	Scoggins Reservoir/ Henry Hagg Lake	Stored Water Secondary	2/20/1963	-	-	500	Bureau of Reclamation	Municipal	-	-
	ASR LL-019	-	Sain Creek, Tualatin River, Scoggins Creek and the Bull Run River, tributaries of Scoggins Creek, the Willamette River, the Tualatin River, and the Sandy River.	All JWC Sources	-	Recovery: up to 28,000 gpm (40.3 mgd) total (2000 gpm, 2.9 mgd) from each of 14 wells	-	2.1 billion gallons	Joint Water Commission	ASR	9/27/2021	-

Note:
Abbreviation: cfs – cubic feet per second; gpm – gallons per minute.

5.3.1 Direct Diversion (i.e., Natural Flow) Water Rights

JWC manages water rights for the use of natural flow from the Tualatin River, Sain Creek, Scoggins Creek, and Gales Creek for diversion at Spring Hill Pumping Plant (SHPP).

These water rights for natural flow consist of six certificates, one water-right-in-transfer status, one permit, and one permit application. Regardless of the source, all water rights authorize JWC to divert water from the SHPP on the Tualatin River.

The rate of water available under JWC's natural flow water rights during the summer's low flow period ranges from 0 to 14 cubic feet per second (cfs) (i.e., 0 to 9 mgd). Meanwhile, the rate of water available during the winter fluctuates depending on flow conditions but is typically available under all JWC natural flow water rights, with the exception of Permit S-54737 for the use of water from Scoggins Creek, which is subordinate to Scoggins Reservoir's fill schedule.

Permit S-54737 authorizes the use of up to 75 cfs. The water available under Permit S-54737 was limited to 26 cfs from 2011 through 2020 by condition of the Oregon Water Resources Department's (OWRD's) order approving the JWC's 2010 Water Management and Conservation Plan (WMCP). However, the full authorized rate of 75 cfs under Permit S-54737 would have been available 47 percent of the time from December through March, with approximately 61 percent of the maximum authorized volume of water available due to insufficient flow in Scoggins Creek. In recent years, the water available under this permit has declined due to decreased fall and winter precipitation, which delays the refill of Hagg Lake. In general, water is rarely available under Permit S-54737 during May, October, and November, given the demand for other uses, including irrigation, water quality, and storage for the refill of Scoggins Reservoir.

The rate of JWC's other existing natural flow water rights for the Tualatin River and its tributaries is 119.5 cfs (i.e., 77.2 mgd). Application S-88506, which requests the use of water from the Tualatin River in parallel to but not additive to Permit S-54737, would not increase this rate, but would increase the reliability of the JWC's water rights during the winter months. The resulting reliable winter supply is expected to be 163.5 cfs (i.e., 105.6 mgd).

Note that the OWRD has not yet issued a permit for Application S-88506. This topic is further discussed in Section 5.3.2.1.

5.3.2 Storage Rights and Secondary Rights to Use Stored Water

5.3.2.1 Barney Reservoir

Certificates 81023 and 81024, combined, authorize the storage of up to 20,000 ac-ft of water from the Middle Fork of the North Fork Trask River in Barney Reservoir. The reservoir's stored water is currently allocated among the JWC partners and Clean Water Services via agreements through the BRJOC, and some of the water is also released to the Trask River watershed on behalf of the Oregon Department of Fish and Wildlife (ODFW). Up to 14,886 ac-ft, or 4,851 million gallons (MG), can be diverted at the SHPP for municipal purposes.

Certificates 81020 and 81022 and Permit S-55219 authorize the use of the water stored in Barney Reservoir for municipal purposes. Certificate 81020 is limited to 38.7 cfs with no volume limit while Certificate 81022 authorizes the use of up to 500 ac-ft with no rate limit. Finally, Permit S-55219 authorizes the use of up to 8,734 ac-ft at a rate of up to 30 cfs.

Note that Certificate 81020 authorizes the use of natural flow from the Middle Fork of the North Fork Trask River in addition to stored water from Barney Reservoir. In theory, this water right allows the diversion of water from Barney Reservoir to the Tualatin River for municipal use without requiring the water to be stored in Barney Reservoir. In practice, however, the natural flow portion of this water right is managed as a secondary water right for Barney Reservoir.

5.3.2.2 Scoggins Reservoir

The Bureau of Reclamation holds Certificate 81149, which authorizes the storage of up to 60,000 ac-ft from Scoggins Creek in Scoggins Reservoir/Henry Hagg Lake for irrigation, municipal, water quality, fish, and recreation purposes. The Bureau of Reclamation has contracts with Hillsboro, Forest Grove, and Beaverton to provide a combined total of up to 13,500 ac-ft of stored water from the reservoir for municipal purposes under two secondary water rights.

Additionally, Certificate 87304 authorizes the use of up to 70 cfs and up to 13,000 ac-ft from Scoggins Reservoir for municipal purposes while Certificate 93873 authorizes the use of up to 500 ac-ft of water stored from Scoggins Reservoir for municipal purposes during each year's irrigation season.

5.4 Demand Forecast

JWC's water supply year has the following two seasons, for which demand and available sources of supply differ:

- **Non-peak season**: November through April.
- **Peak season**: May through October.

This section forecasts the total non-peak-season and peak-season demand that will be placed on JWC and the WTP by their partners and wholesale customers, according to each of these entities' respective projections.

5.4.1 Forecast Assumptions and Methodology

JWC's demand projections compile each partner and wholesale customer's individual demand projections. More precisely, JWC's demand forecast methodology is identical to that described in the *2020 JWC Water Management and Conservation Plan*, and summarized in section 5.4.2.1 below, with three exceptions:

- Demand projections for Forest Grove have been updated since the WMCP was drafted.
- In this master plan, JWC partners' projected peak season demands have been separated into demands expected to be met by JWC and non-JWC sources.
- ASR injection is expected be 55 percent of the full authorized rate of ASR LL-002, LL-019, and LL-027 by 2030, and 100 percent by 2040 (the WMCP assumed 100 percent build-out by 2040).

Table 5.4 shows the sources of information used to confirm each partner and wholesale customer's demand projections.

Table 5.4 Sources for Demand Forecast

Member Agency/ Water User	Demand Forecast Source
Hillsboro	2018 Demand Projections Update Report
Forest Grove	City of Forest Grove Water System Master Plan (February 2020 Draft)
Beaverton	2018 Water System Master Plan
TVWD	2018 Water Master Plan Update
Wholesale	North Plains Water Master Plan + Lutheran School

5.4.2 JWC Demand Forecast

5.4.2.1 Non-Peak Season Total Demand Forecast

JWC's non-peak season demand forecast considered yearly average day demand (ADD), water demands for ASR injections, variability in non-peak season demand, and their daily diversions at the SHPP intake. The non-peak season demand forecast does not distinguish between demands on JWC and non-JWC sources, reflecting water needed if the JWC source were to meet all partner demands during the non-peak season.

The combined total ADD of each JWC partner and wholesale customer was added to the maximum injection rate authorized for ASR LL-002, ASR LL-019, and ASR LL-027. Next, a multiplier of 0.9 was applied to reflect the fact that maximum day demand (MDD) from December through April has historically occurred in December and been approximately 90 percent of the ADD. Finally, an analysis of diversion data was used to generate a peaking factor of 1.15, which was applied to reflect the variability of JWC's diversions at the SHPP intake throughout the course of a day. Between the 0.9 and 1.15 multipliers, the maximum demand anticipated during the non-peak season is 1.035 times the ADD.

Table 5.5 presents the factors accounted for when calculating the adjusted ADD anticipated during JWC's non-peak season, which is projected to be 162.61 cfs (i.e., 105.11 mgd) in 2030 and 178.79 cfs (i.e., 115.57 mgd) in 2040. Figure 5.7 forecasts combined non-peak season demands and the development of existing water rights from 2020 through 2040.

Table 5.5 Non-Peak Season Projected Demand (JWC and non-JWC sources), 2030 and 2040

Demand Factors	2030 ADD (mgd)	2030 ADD (cfs)	2040 ADD (mgd)	2040 ADD (cfs)
Hillsboro	28.73	44.45	34.51	53.40
Forest Grove	5.36	8.30	6.84	10.59
Beaverton	10.52	16.28	11.12	17.21
TVWD	26.41	40.86	28.62	44.28
Wholesale	0.71	1.10	1.17	1.81
User Total	**71.73**	**110.98**	**82.27**	**127.28**
ASR LL-002 Max Injection	6.88	10.64	12.5	19.34
ASR LL-019 Max Injection	6.41	9.92	11.66	18.04
ASR LL-027 Max Injection	3.035	4.69	5.51	8.53
ASR Total	**16.32**	**25.25**	**29.67**	**45.91**
User + ASR Totals (Total ADD)	**88.05**	**136.23**	**111.94**	**173.19**
90% of Total ADD (Dec)	**79.24**	**122.61**	**100.74**	**155.87**
Total with WTP Daily PF (1.15)	91.13	141.00	**115.85**	**179.25**

Note:
Abbreviation: PF – peaking factor.

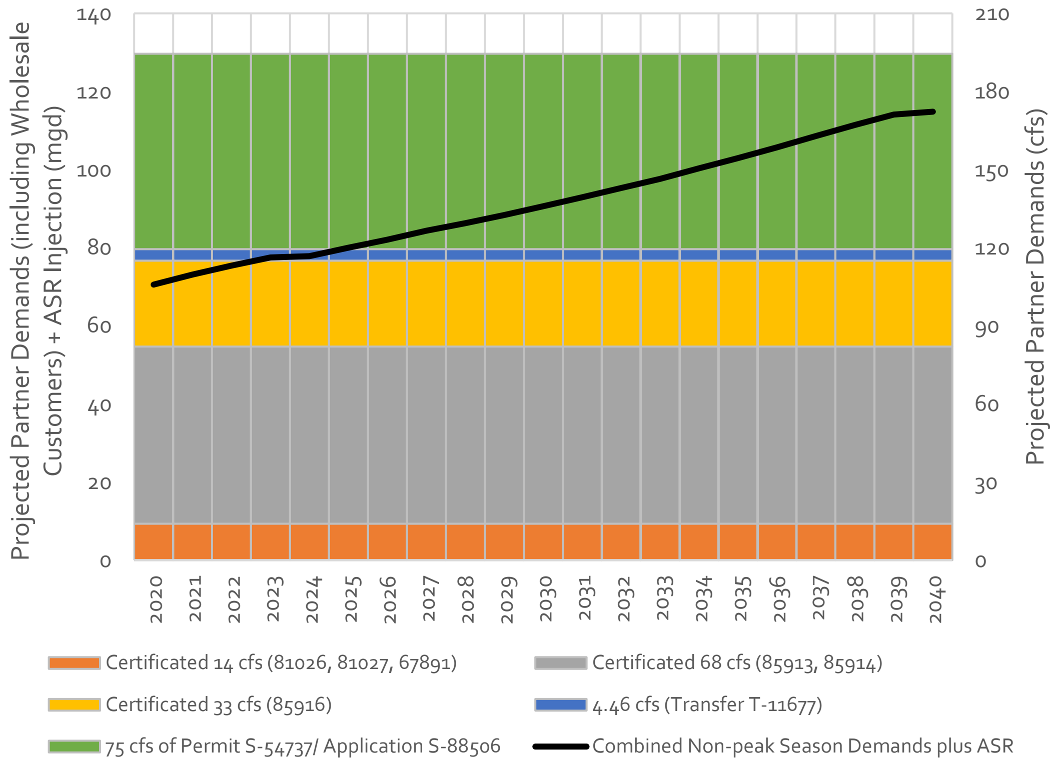


Figure 5.7 Combined Projected Non-Peak Season Demands and Development of Existing Water Rights from 2020 through 2040

Consistent with the extension-of-time approval for Permit S-54737, the JWC partners project non-peak season demands of up to 125 mgd prior to 2071. The partners will continue to refine demand projections and evaluate sources of supply available to regularly meet their demands.

As described in Section 5.2.2, the OWRD has not yet issued a water use permit for Application S-88506. JWC's reliable non-peak season water supply without the requested permit is 77.2 mgd.

5.4.2.2 Peak Season JWC WTP Demand Forecast

JWC partners developed JWC WTP projections for their peak demands through 2042. The methodology to produce these projections is consistent with that described above, only the partners identified portions of their total demands that they plan on fulfilling using the JWC supply for this analysis.

Table 5.6 shows the projected demands for each JWC partner and North Plains as well as the total projected demand placed on the JWC WTP.

Table 5.6 JWC Partner Demand Projections for the JWC WTP through 2042

Year	Beaverton MDD (mgd)	Hillsboro MDD (mgd)	Forest Grove MDD (mgd)	TVWD MDD (mgd)	North Plains[(1)] MDD (mgd)	Total MDD (mgd)
2021	12.82	38.22	5.80	14.50	1.20	72.54
2022	11.85	40.01	6.31	14.50	1.20	73.87
2023	13.31	41.20	6.82	14.50	1.20	77.02
2024	13.65	41.75	7.33	14.50	1.20	78.43
2025	13.88	41.75	7.70	14.50	1.50	79.33
2026	14.09	41.75	8.08	14.50	1.50	79.91
2027	11.78	41.75	8.45	14.50	1.50	77.99
2028	11.32	41.75	8.83	14.50	1.50	77.90
2029	10.62	41.75	9.21	14.50	1.50	77.57
2030	10.70	41.75	9.53	14.50	2.00	78.47
2031	10.77	41.75	9.85	14.50	2.00	78.87
2032	8.95	41.75	10.17	14.50	2.00	77.37
2033	9.02	41.75	10.49	14.50	2.00	77.77
2034	9.10	41.75	10.82	14.50	2.00	78.16
2035	9.17	41.75	11.14	14.50	2.50	79.06
2036	9.25	41.75	11.46	14.50	2.50	79.46
2037	9.33	41.75	11.78	14.50	2.50	79.86
2038	10.01	41.75	12.10	14.50	2.50	80.86
2039	10.37	41.75	12.43	14.50	2.50	81.55
2040	10.73	41.75	12.47	14.50	2.50	81.95
2041	11.09	41.75	12.52	14.50	2.50	82.36
2042	11.45	41.75	12.56	14.50	2.50	82.77

Note:

(1) North Plains' demand is shown in this table as an additive demand to JWC partner demands. Since North Plains does not have an ownership allocation of the WTP, their demand is served out of a portion of JWC partners' allocations. While total demand remains below WTP capacity, it is assumed that North Plains' demands can be served out of whichever JWC partner is not utilizing their full allocation.

Beaverton's peak demand from the JWC WTP is expected to vary between approximately 9 and 14 mgd through 2042. While Beaverton expects that their total demand on JWC supply will increase through the next several decades, they also expect to use ASR to meet peak demands. By storing JWC-supplied water in the aquifer during times of high supply and low demand (i.e., the non-peak season) and then withdrawing water from the aquifer during peak demand periods, Beaverton's peak season demand on the WTP is projected to stay relatively constant and may even decrease over time. When JWC nears the SHPP intake and WTP capacity of 85 mgd, Beaverton plans to participate in a 5 mgd expansion to improve the redundancy and resiliency of their community water supply.

Hillsboro projects that their demand from the JWC WTP will slightly increase over the next several years, after which they will maximize the use of their full allocation through 2042. While Hillsboro expects that their total demand will exceed their JWC WTP allocation, that additional demand is expected to be met by the Willamette Water Supply System starting in 2026.

Forest Grove expects their demand on the JWC WTP to essentially double from 6 mgd in 2021 to nearly 13 mgd in 2042. They expect to exceed their current 10-mgd allocation by 2032.

TVWD is projected to continue maximizing the full use of their allocation of 14.5 mgd through the extent of the planning period. Since TVWD's full demand currently exceeds their allocation from JWC and is expected to continue doing so, they will continue to utilize other water supply sources to meet that additional demand.

Finally, North Plains, per their agreement with JWC, is expected to increase their water purchase from 1.2 mgd in 2021 to 2.5 mgd in 2042. The small amount of water sold to the Westside Lutheran School is expected to stay constant.

As shown in Figure 5.8, the JWC WTP's current capacity of 85 mgd is expected to be sufficient to meet demands through 2042, though individual partners may require more than their current allocation before then. For example, Forest Grove is currently projecting to exceed their 10 mgd allocation by 2032, but only by approximately 1 to 3 mgd until 2042. Within the next 20 years, JWC anticipates meeting this additional demand through leasing unused capacity to partners who need this supply rather than undertaking a capital improvement project to expand the intake capacity, via the SHPP or other new facility, and the WTP's total capacity.

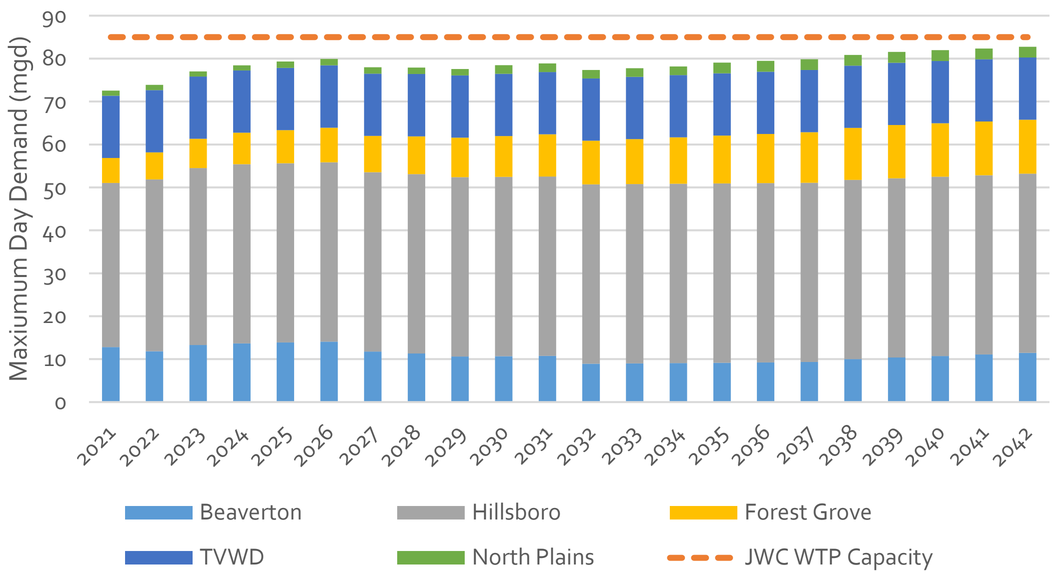


Figure 5.8 JWC WTP Partner Demand Projections through 2042

The anticipated demand is projected to near JWC's current 85-mgd treatment capacity by the end of the planning period, with total demand projected to be approximately 83 mgd in 2042. Demand projections should be re-evaluated with each master plan update, scheduled to occur at least every 10 years, to ensure that capacity will be sufficient to meet demands within the planning period or to allow enough time to plan and implement any needed expansions.

Chapter 6

INTAKE EVALUATION

6.1 Introduction

The Joint Water Commission (JWC) receives their entire water supply from one existing intake on the Tualatin River, the Spring Hill Pumping Plant (SHPP). Owned by the United States Bureau of Reclamation (USBOR), the SHPP is located near the JWC Water Treatment Plant (WTP) and used by both JWC and the Tualatin Valley Irrigation District (TVID).

Although JWC currently has a total intake pumping capacity of 88 million gallons per day (mgd), they anticipate their maximum day water demand to surpass this capacity by year 2050. Additionally, the SHPP is not seismically resilient at this time. Upgrades to enhance its safety and reliability through potential, future earthquake events would require coordination with and approval by the USBOR.

JWC is now considering an intake project that develops one or more intakes to help meet their build-out water supply capacity of 125 mgd while also providing at least 44 mgd of a seismically resilient water supply. The intakes, including the SHPP, must provide an initial firm capacity of 88 mgd to meet JWC’s supply needs for the 20-year planning period, an interim firm capacity of 105 mgd, and a build-out firm capacity of 125 mgd under all river flow conditions. At this time, JWC does not know when the interim or build-out capacities will be required.

Whether developing a new intake or upgrading the existing SHPP to withdraw more water from the Tualatin River, at least one intake must be located along the Tualatin River in the vicinity of the JWC WTP to divert all of JWC's existing water rights and minimize raw water piping to the WTP. Since the early 2000s, JWC has considered building a raw water pipeline from Scoggins Dam to the JWC WTP, but their existing water rights limit the flow that can be supplied from this dam. Because the allowable flow from Scoggins Dam alone does not meet JWC’s supply needs, supply from a raw water pipeline is considered supplemental to intakes on the Tualatin River.

This chapter conducts an alternatives analysis of improvements that allow JWC to meet their partners’ future water demands as well as their seismic level of service (LOS) goals as introduced in Chapter 3. Tasked with the analysis, Carollo Engineers (Carollo) first assessed three viable alternatives for a seismically resilient intake on the Tualatin River and then evaluated the benefits and challenges of building a new, supplemental raw water pipeline from Scoggins Dam to the JWC WTP.

Figure 6.1 shows the SHPP, and the two properties considered as sites for a new Tualatin River intake. The map also shows the 6.5-mile raw water pipeline route from Scoggins Dam to the JWC WTP that was proposed in the Murraysmith and CH2M Hill 2004 report *Raw Water Pipeline Routing Analysis and Preliminary Environmental Review Report* (2004 Report).

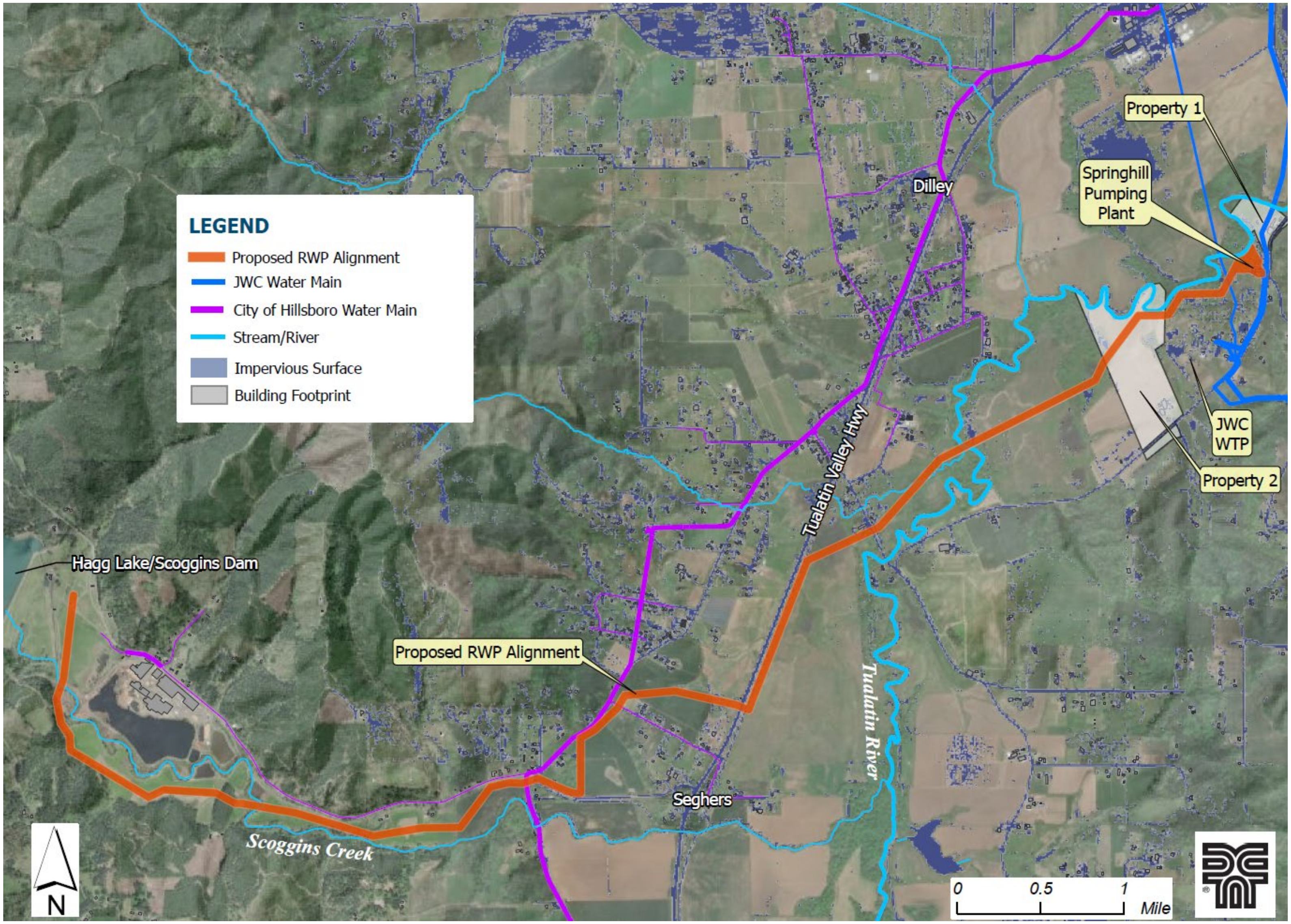


Figure 6.1 Aerial View of Proposed Raw Water Pipeline Route, Intake Properties, and Terrestrial Habitats

6.2 Permitting Considerations

This section reviews permitting considerations that are applicable to both the development of a new intake on the Tualatin River and construction of a raw water pipeline from Scoggins Dam.

6.2.1 Federal and State Permitting Considerations

Federal and state permitting is triggered when a facility or project is found to affect resources regulated by federal or state agencies, such as wetlands, endangered species, or migratory birds.

Discussed per regulatory agency, the following sections summarize relevant environmental rules and regulations that pertain to this project. Environmental permitting factors specific to the alternatives are discussed in their respective sections.

6.2.1.1 U.S. Army Corps of Engineers and Oregon Department of State Lands

Under Section 404 of the Clean Water Act, the U.S. Army Corps of Engineers (USACE) has jurisdiction over fill within wetlands and below the ordinary high water (OHW) of any river or stream in the U.S. USACE also has jurisdiction within and below any "navigable" waterway in the U.S., per Section 10 of the Rivers and Harbors Act. Meanwhile, the Oregon Department of State Lands (DSL) has jurisdiction over fill or removal within wetlands and below the OHW line of Oregon’s waterways, per the Oregon Removal-Fill Law.

Both USACE and DSL require project proponents to first avoid impacts on regulated resources and, if avoidance is not practicable, minimize those impacts by restoring the territory to pre-project or better conditions and mitigate permanent impacts (i.e., conversion of wetlands to upland conditions or impacts that last longer than two years). If impacts cannot be avoided, a joint permit application (JPA) must be submitted to obtain permits from the two agencies.

The following additional investigations and documentation are often required in JPA submittals:

- Wetland delineation report.
- Wetland functional assessments for permanently impacted wetlands only.
- Stream functional assessments for permanently impacted streams only.
- Design plans (30 to 60 percent design level is typically sufficient).
- Compensatory mitigation plans (i.e., written report and design sheets) or documentation of a mitigation bank credit purchase.
- Site-restoration design plans for temporary impacts.

Additionally, if a project must obtain a Section 404 permit from the USACE, the permit must comply with the federal Endangered Species Act (ESA), Section 106 of the National Historic Preservation Act, and 401 Water Quality Certification, all of which are described in the following subsections.

Whether placing a new intake on the Tualatin River or upgrading the existing SHPP, JWC will require permits from USACE and DSL. These intake sites and associated pipeline connections have a high potential of affecting wetlands and waters that cannot be avoided, and mitigation for permanent impacts will be required.

6.2.1.2 National Marine Fisheries Service

The National Marine Fisheries Service (NMFS) evaluates a project's impacts on anadromous fish species protected under the federal ESA. The following listed anadromous fish reside within the Tualatin Basin that may occur within the project area:

- Upper Willamette River Chinook salmon (*Oncorhynchus tshawytscha*).
- Upper Willamette River Steelhead (*O. mykiss*).

Wetland or waterway impacts trigger a Section 404 permit and compliance with the ESA. While there is currently no regulatory requirement, potential floodplain impacts and the need to consult with the Federal Emergency Management Agency (FEMA) may one day trigger ESA compliance. Because the intake project is intended to help JWC meet their build-out water supply, it will aim to increase JWC's ability to withdraw water from the Tualatin River; therefore, NMFS must assess impacts on listed fish from downstream of the project to the mouth of the Columbia River through a formal ESA consultation. This consultation will require JWC to work closely with NMFS in preparing a Biological Assessment of potential impacts on ESA-listed fish.

Through the consultation, NMFS will evaluate any increase of impervious surface greater than 1 acre. The project must meet the requirements of Standard Local Operating Procedures for Endangered Species (SLOPES) for stormwater management to avoid adversely affecting listed fish at the project site and downstream. Should NMFS confirm that the project will not jeopardize the identified species or their habitats, they will issue a Biological Opinion with terms and conditions and reasonable and prudent measures to minimize any project-related impacts to listed fish at the project sites and downstream.

6.2.1.3 U.S. Fish and Wildlife Service

USFWS is required to evaluate project impacts on species protected under the federal ESA. Figure 6.2 shows an aerial photograph of the aquatic habitats and potential wetland areas that fall within the intake project's vicinity, which primarily consists of rural and agricultural land uses.

Several listed plant and wildlife species are noted in the general project vicinity (IPaC database search, USFWS 2021), including Kincaid's Lupine (*Lupinus sulphureus spp. kincaidii*), Nelson's Checker-mallow (*Sidalcea nelsoniana*), Willamette Daisy (*Erigeron decumbens*), Bradshaw's lomatium (*Lomatium bradshawii*), Water Howellia (*Howellia aquatilis*), and Streaked Horned Lark (*Eremophila alpestris strigata*).

Although unlikely, the intake locations have conditions with the potential to support these species. As such, a biologist must confirm the presence or absence of suitable habitats for these species. If such habitats exist, then surveys are recommended prior to any ground-disturbing activities and as part of the consultation with USFWS, per the Section 404 permitting process.

The Section 404 permit also requires consultations for the Migratory Bird Treaty Act (MBTA) and the Bald and Golden Eagle Protection Act, which are also regulated by the USFWS per the Fish and Wildlife Coordination Act. Under these acts, "take" of migratory birds and/or eagles is regulated. Removal of trees with active bird nests can result in "take"; as such, any tree removal that is demanded to accommodate the intake and pipeline improvements must comply with the MBTA and Bald and Golden Eagle Protection Act.

Specific to the project area, the presence of tall vegetation surrounding the intake properties and their relatively small size may limit potential use by the Streaked Horned Lark. If trees must be removed, biologists can work with the projects' design and construction teams to avoid or minimize impacts to the extent practical.

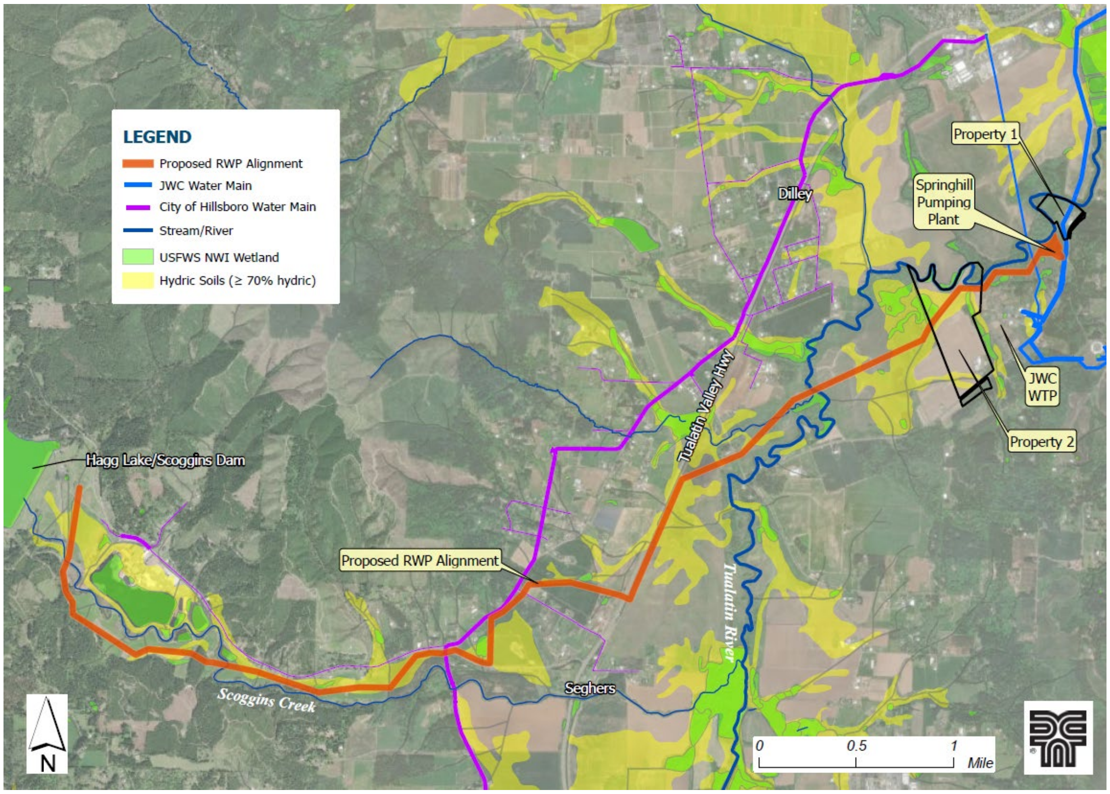


Figure 6.2 Aquatic Habitats and Wetlands

6.2.1.4 Federal Emergency Management Agency

Figure 6.5 shows that portions of JWC's project area fall within various FEMA mapped floodplains and floodways associated with the Tualatin River and Scoggins Creek. Projects that create new impoundments or other hydraulic changes in a designated floodplain or floodway and are unable to demonstrate a 'no net rise' in the Base Flood Elevation (BFE, defined as the surface water elevation resulting from a 1 percent chance annual flood, or 100-year flood) must be authorized by FEMA under one of the following processes:

Revision of a Flood Insurance Rate Map (FIRM)

Physical Map Revision (PMR) or Letter of Map Revision (LOMR) for a change to identified Special Flood Hazard Areas (SFHAs), regulatory floodway designations , BFE and/or planimetric feature(s). A PMR is an action whereby one or more map panels are physically revised and republished. A LOMR is a document that officially revises a portion of the effective FEMA flood map via letter without physically revising and reprinting the entire map panel.

No Revision to a FIRM, but a Letter Confirming the Project, if Built as Proposed, would be Recognized by FEMA

A Conditional Letter of Map Revision (CLOMR) is FEMA's comment on a proposed project that would, upon construction, affect the hydrologic or hydraulic characteristics of a flooding source and thus result in the modification of the existing regulatory floodway, the effective BFEs or the SFHA.

Because of the project's location within the floodplain and regulatory floodway, placing a new intake on the Tualatin River and/or modifying the existing SHPP is expected to trigger the need to evaluate 'no net rise' by modeling upstream and downstream flood elevation changes that would result from the project. 'No net rise' can sometimes be achieved with a balance of cut-and-fill material; however, the volume of the intake and other potential modifications within the floodway make the balance unlikely and a LOMR would be the most likely process to achieve FEMA approval.

6.2.1.5 Oregon Department of Fish and Wildlife

The Oregon Department of Fish and Wildlife (ODFW) is tasked with managing Oregon's native fish and wildlife and their habitats. Native migratory fish that are found in the Tualatin River and Scoggins Creek include native Chinook and steelhead, cutthroat trout (*Oncorhynchus clarkii*), and Pacific lamprey (*Entosphenus tridentatus*). ODFW has the opportunity to provide input on a project through several state regulatory processes:

- DSL solicits ODFW input through the Removal-Fill permitting process.
- Oregon Water Resources Department (OWRD) will solicit ODFW input on water rights authorizations with the potential to impact native migratory fish.
- Fish screen structures must meet ODFW fish screen criteria or receive a fish screen waiver.
- ODFW will review through Oregon's fish passage regulations.

Under Oregon's Fish Passage regulations (Oregon Revised Statute [ORS] 509.580 through 910 and OAR 635, Division 412) the owner or operator of an artificial obstruction located in waters in which native migratory fish are currently or were historically present must address fish passage requirements prior to certain trigger events, such as installation or replacement of an obstruction. Addressing fish passage requirements requires the owner/operator to obtain from ODFW: 1) approval of a Fish Passage Plan demonstrating how passage will be provided, 2) a waiver from providing passage, or 3) an exemption from providing passage. It is the intent of ODFW that option #1 should be sought and passage should be provided at the artificial obstruction.

6.2.1.6 Oregon Department of Environmental Quality

Per Section 401 of the Clean Water Act, the Oregon Department of Environmental Quality's (DEQ) 401 Water Quality Certification program reviews and evaluates the water quality impacts of projects requiring a federal permit or license to conduct activities that may result in discharges, including those of dredge and fill material, in U.S. waters. If a Section 404 permit is required for the project, a 401 certification from DEQ will also be required.

Since the intake project will increase JWC's ability to withdraw water from the Tualatin River and likely pose impacts on wetlands and streams, DEQ will evaluate the effort's potential water quality impacts. According to recent water-supply and withdrawal projects in the Willamette River Basin, DEQ may require the preparation and submittal of a thermal trading plan to address potential influences over river temperature. DEQ-authorized thermal trading plans allow facilities to meet regulatory obligations by:

- Purchasing equivalent or larger pollution reductions from another source; or

- Protecting or restoring riparian areas, wetlands, floodplains, and aquatic habitats to mitigate temperature impacts.

6.2.1.7 State Historic Preservation Office

If the project requires a Section 404 permit, it will require authorization from Section 106 of the National Historic Preservation Act. This regulation calls for the consideration of project effects on historical properties and cultural resources that are determined eligible or potentially eligible for listing under the National Register of Historic Places. To this end, the project will require government-to-government consultation with the State Historic Preservation Office and appropriate federally recognized tribes.

Since a Section 404 permit is likely required for all of the considered alternatives, JWC is recommended to contract a qualified archaeologist or historic specialist to determine the potential for cultural resources to occur within the project area. A Section 106 consultation, if required, is conducted in parallel with the Section 404 permitting process.

6.2.2 Local Permitting Considerations

6.2.2.1 Base Zoning and Overlays

The intake alternatives for the Tualatin River are all located in rural unincorporated Washington County, outside the urban growth boundary. As shown in Figure 6.3, all land use zones in the analyzed intake areas consist of rural designations that are intended to protect rural uses such as farms, forests, and agriculture. All of the intake alternatives are located within the Exclusive Farm Use (EFU) zone and must comply with the provisions of Chapter 340 of the Washington County Community Development Code (CDC). Properties on the Tualatin River and Scoggins Creek also have mapped Significant Natural Resource (SNR) designations, which are regulated by Chapter 422 of the Washington County CDC. Chapter 422 implements Oregon Statewide Planning Goal 5: Natural Resources, Scenic and Historic Areas, and Open Spaces as well as the Riparian Corridor protections in the Metro Urban Growth Management Functional Plan Title 3 (categorized as Classes I, II, and III) and the Upland Habitat protections (categorized as Classes A, B, and C) under Title 13 of the Metro Urban Growth Management Functional Plan. Figure 6.4 shows Goal 5 resource categories and Title 13 habitat protections.

Based on the nature and anticipated location of the infrastructure that would be proposed under any of the intake alternatives, the SNR designations identified in Washington County CDC Chapter 422 that would require analysis likely include the following:

- **422-2.1 Water Areas and Wetlands:** 100-year floodplain, drainage hazard areas and ponds, except those already developed.
- **422-2.2 Water Areas and Wetlands and Fish and Wildlife Habitat:** Water areas and wetlands that are also fish and wildlife habitat.

In general, Chapter 422 requires minimum setbacks from the river and preservation of a natural vegetation buffer along the river. Setbacks, buffer width, and other standards are dependent on actual site conditions and the proposed development. Field investigations will be required to review site conditions and determine the extent of the SNR areas prior to development of a proposed project.

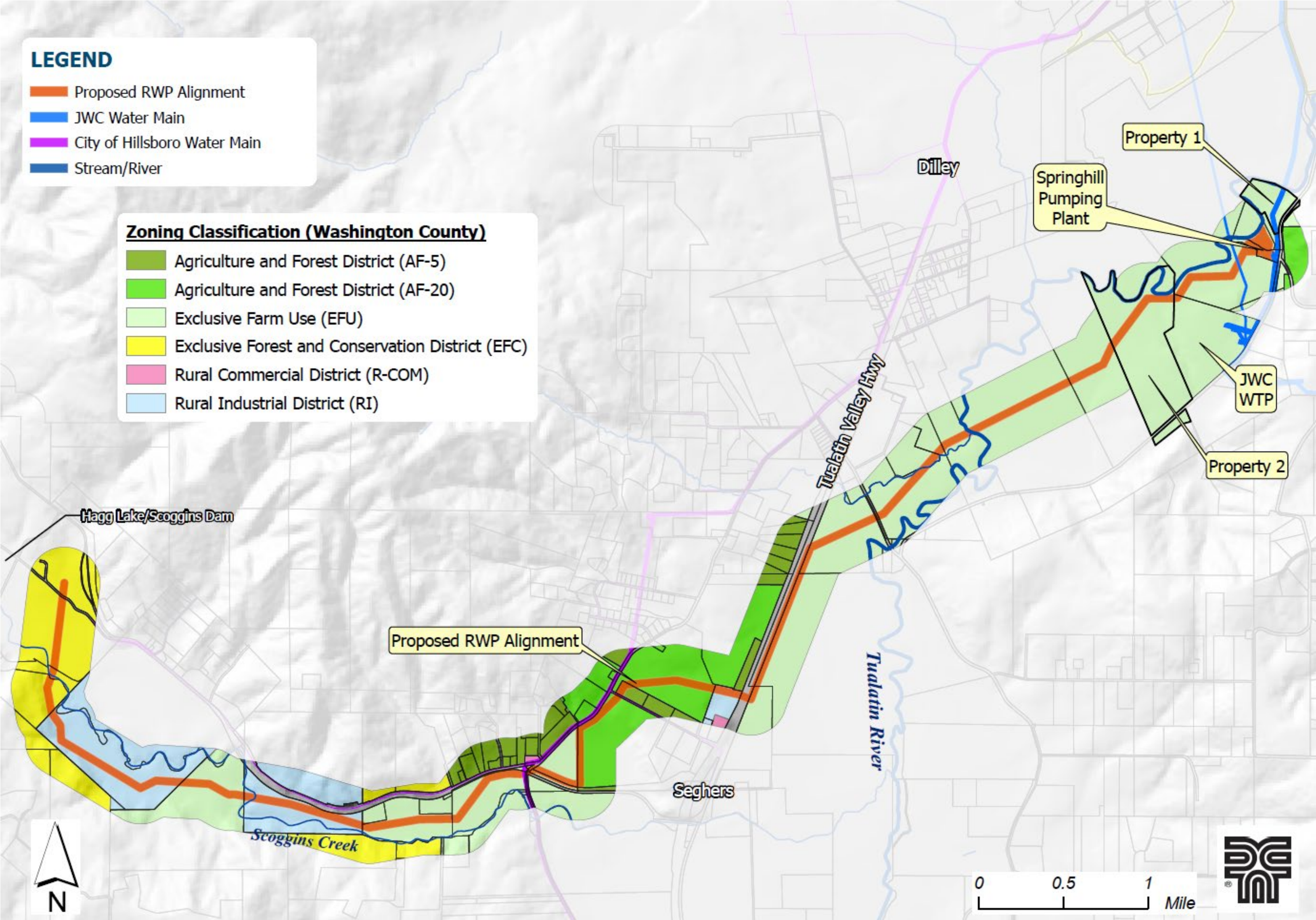


Figure 6.3 Washington County Zoning Map

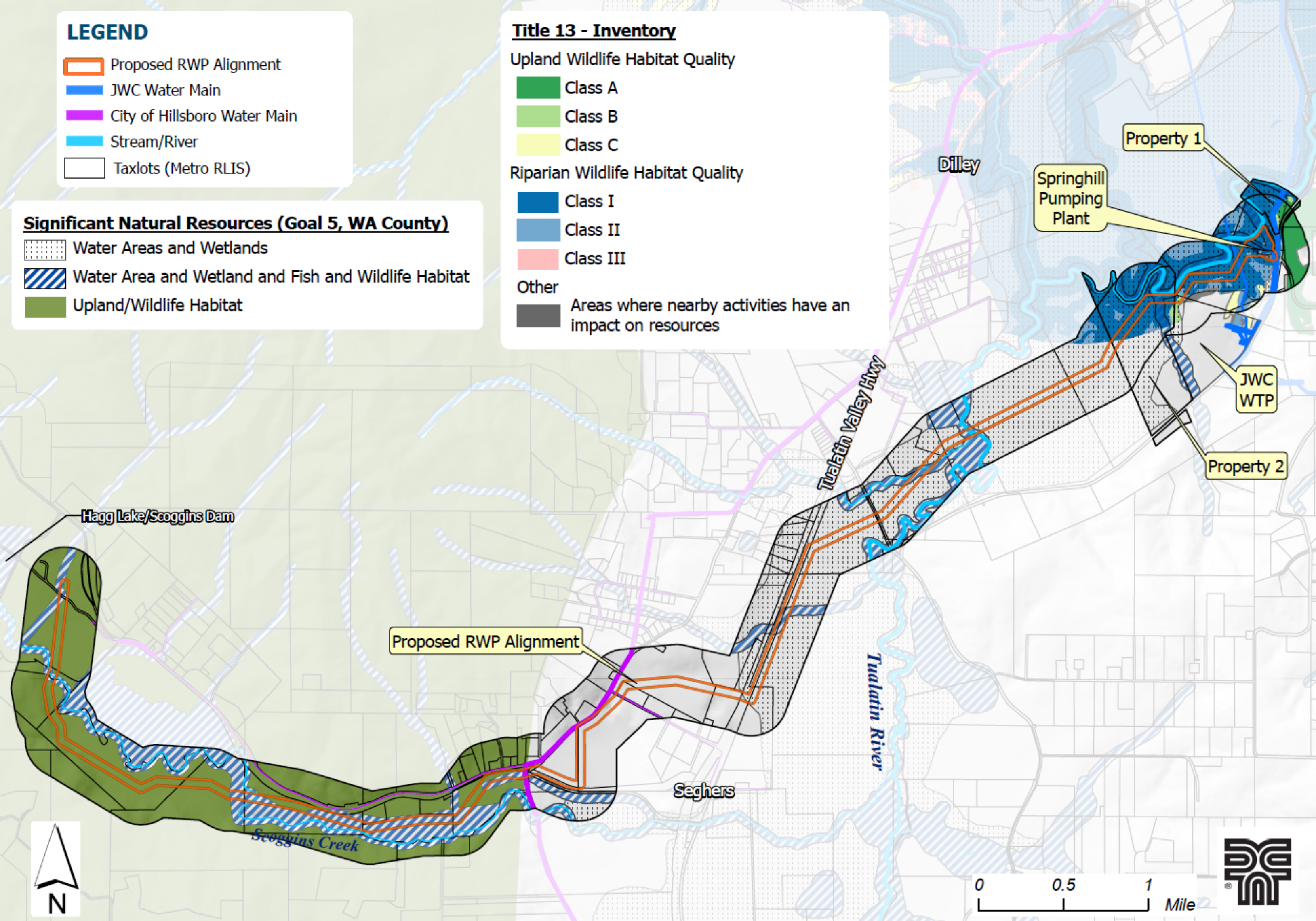


Figure 6.4 Significant Natural Resources and Goal 5 Resources Map

6.2.2.2 Floodplains

Any proposed activity in a floodplain or regulatory floodway will require a permit from Washington County. Figure 6.5 shows that portions of JWC's project area fall within various FEMA mapped floodplains and floodways associated with the Tualatin River and Scoggins Creek. Washington County participates in the National Flood Insurance Program (NFIP). As a result, Washington County is required to adopt and enforce floodplain management regulations that help mitigate flooding effects in accordance with NFIP standards. This is accomplished via the standards set forth in Washington County CDC Chapter 421, Floodplain and Drainage Hazard Area Development.

Chapter 421 requires any uses that may be allowable within the floodplain to demonstrate that they will not result in any increase in the BFE. In addition, Chapter 421 requires that new and replacement water supply systems in FEMA-designated 100-year floodplains must be designed to minimize or eliminate the infiltration of floodwaters into those systems. (FEMA recommended best practice is for critical facilities such as water infrastructure to be designed to achieve 500-year flood resilience). In addition to FEMA-designated floodplains, Washington County also regulates "drainage hazard areas" (DHAs) throughout its jurisdiction. The Washington County-regulated DHA corresponds to the 25-year floodplain. Generally, impacts to the DHA will require evaluation in areas adjacent to water resources where flooding may be possible but FEMA has not mapped the 100-year floodplain. If a DHA evaluation is necessary, Washington County requires a delineation of the DHA based on hydrological and hydraulic analysis by an Oregon licensed engineer. In addition to the existing Washington County CDC requirement to address potential floodplain impacts, future projects with potential floodplain impacts may trigger an evaluation of impacts to endangered salmonids, according to pending NFIP updates.

6.2.2.3 Development Review Permit

An intake would be expected to qualify as a "public utility service facility" under the provisions of the Washington County CDC. Section 430-105, Public Utility, defines a "public utility service facility" as one that "includes buildings, structures, and equipment within a fenced or otherwise enclosed area for the purpose of switching, regulating or controlling public utility services" (430-105.3). Within the EFU zone, utility facilities necessary for public service are subject to a Type II Procedure per Section 340-4.1 R of the Washington County CDC.

In order to demonstrate that the intake qualifies as a public utility service facility and is therefore an allowed use in the EFU zone, a development review permit for construction of the intake must address the following components, as defined in Section 430-105.4 A through C:

- **430-105.4 A**: Present or future need for the facility and how the facility fits into the utility's master plan.
- **430-105.4 B**: Minimum area required for the facility for present and anticipated expansion.
- **430-105.4 C**: What measures will be taken to minimize damage to paved roads and natural resources or open space.

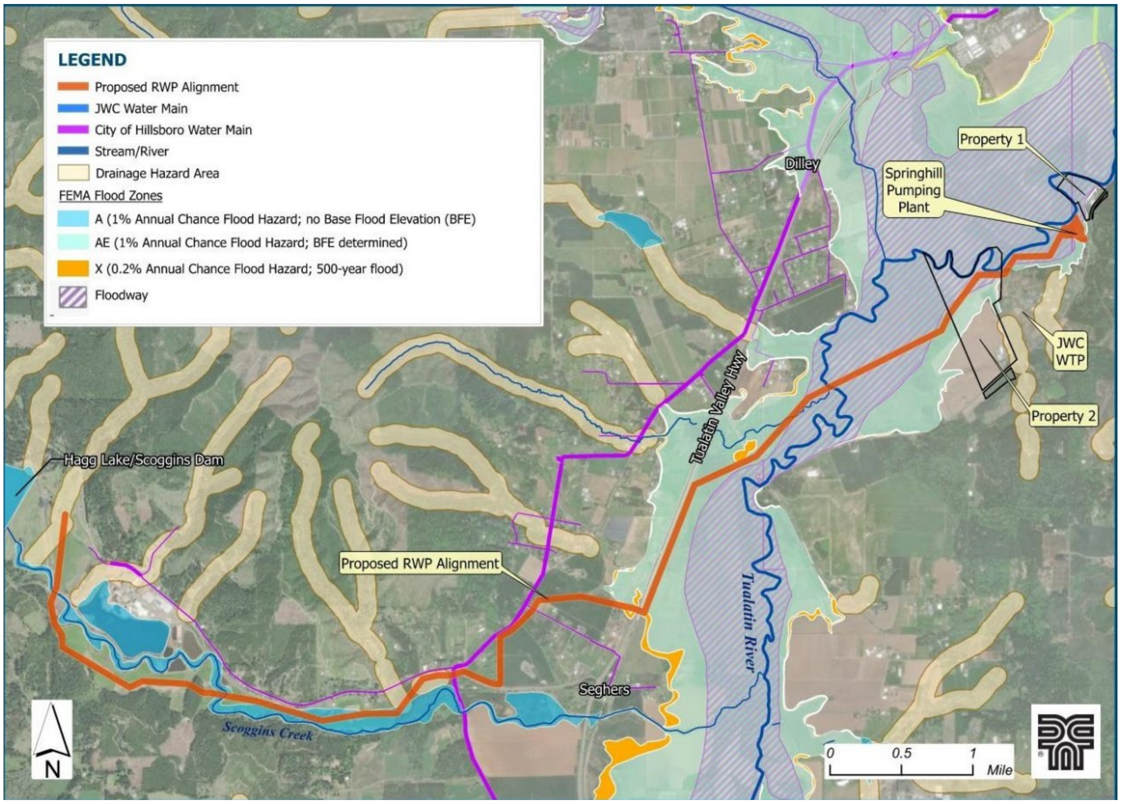


Figure 6.5 Floodplains, Floodways, and Drainage Hazard Areas Map

The Development Review Permit application would also need to address compliance with Washington County CDC Chapters 421, Floodplain and Drainage Hazard Development and 422, Significant Natural Resources. Section 422-3.3 A(3) allows for the installation or construction of utilities, including but not limited to water lines, in the Water Areas and Wetlands and Water Areas and Wetlands and Fish and Wildlife Habitat SNRs. Section 422-3.3 B stipulates that the flood plain and drainage hazard area development criteria must nonetheless be followed.

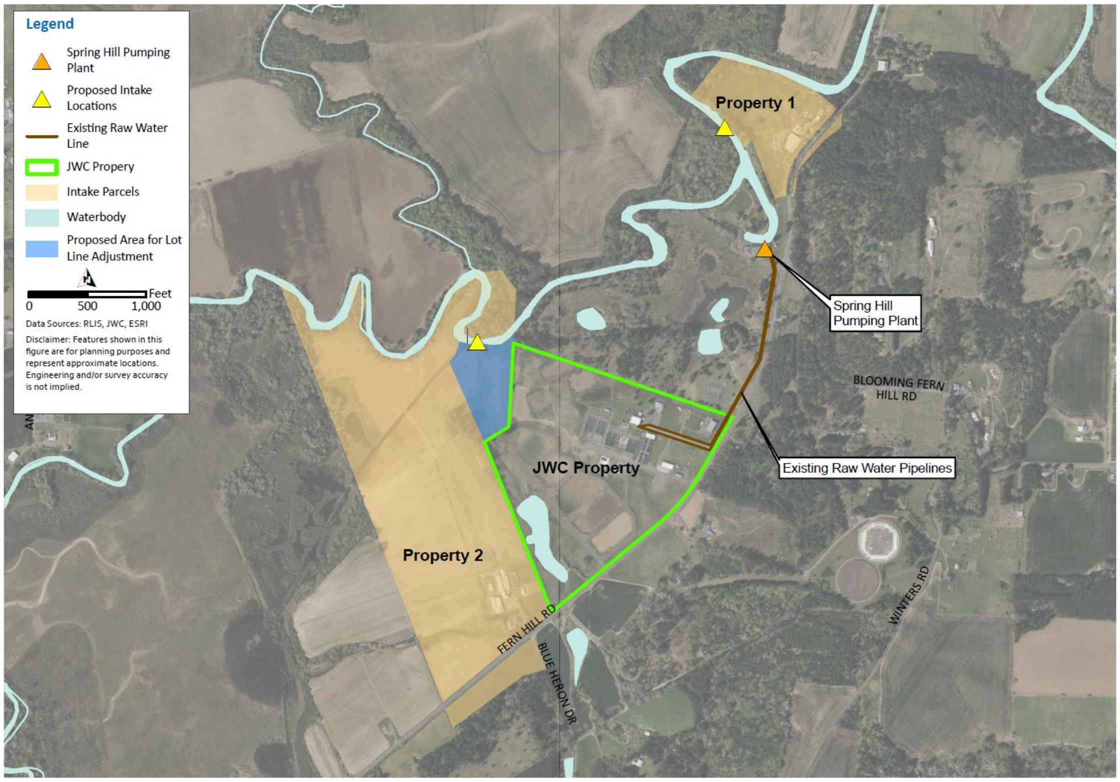
Water supply intakes are not a common use and are not enumerated in Washington County CDC Chapter 421 as a use that is specifically allowed, prohibited, or otherwise restricted in the floodplain. Therefore, permitting an intake in the 100-year floodplain or the regulatory floodway would require coordination between JWC and Washington County's floodplain manager to determine the most reasonably comparable allowable use under Chapter 421.

The intakes could potentially be permitted in the floodplain based on a Type II review process under Section 421-5.11, which allows for "Construction or major improvement or alteration of underground pipes and conduits, including sewer, water and gas lines, transmission and distribution lines for geothermal resources, gas and oil, underground electrical, telephone and television transmission and distribution lines, including necessary accessory structures and drainage systems." Coordination with the Washington County Planning Department's Floodplain Manager would be essential in advance of any proposed project to ensure that the use is appropriately addressed under Section 421-5.11.

6.3 Alternatives Analysis for the Tualatin River Intakes

Under this alternatives analysis, the following three options for intakes along the Tualatin River were compared, assessed, and plotted in Figure 6.6:

- **Property 1**: Develop a new, seismically resilient intake on Property 1.
- **Property 2**: Develop a new, seismically resilient intake on Property 2.
- **Upgrade the SHPP**: Upgrade the capacity and seismic resilience of the existing SHPP. This alternative could be implemented with or without USBOR cost-sharing.



Data Sources: RLIS, JWC, ESRI

Disclaimer: Features shown in this figure are for planning purposes and represent approximate locations. Engineering and/or survey accuracy is not implied.

Figure 6.6 Tualatin River Intake Sites Considered

Even if a new intake is developed on Property 1 or Property 2, JWC is recommended to continue maintaining and using the SHPP as a redundant water supply source for the remainder of its useful life. If the SHPP continues to be used, the minimum initial capacity of the new intake will be 44 mgd to meet JWC's seismic LOS goal. However, the Tualatin River intake alternatives described in this chapter are defined as having a capacity of 88 mgd.

Because Property 2 is adjacent to the JWC WTP property, it may be possible to do a lot line adjustment rather than purchasing the entire property. Figure 6.6 shows the recommended lot line adjustment area for the intake, which consists of approximately six acres.

Any modifications to the SHPP, or modifications to or replacement of the existing rock weir, will involve coordination with USBOR.

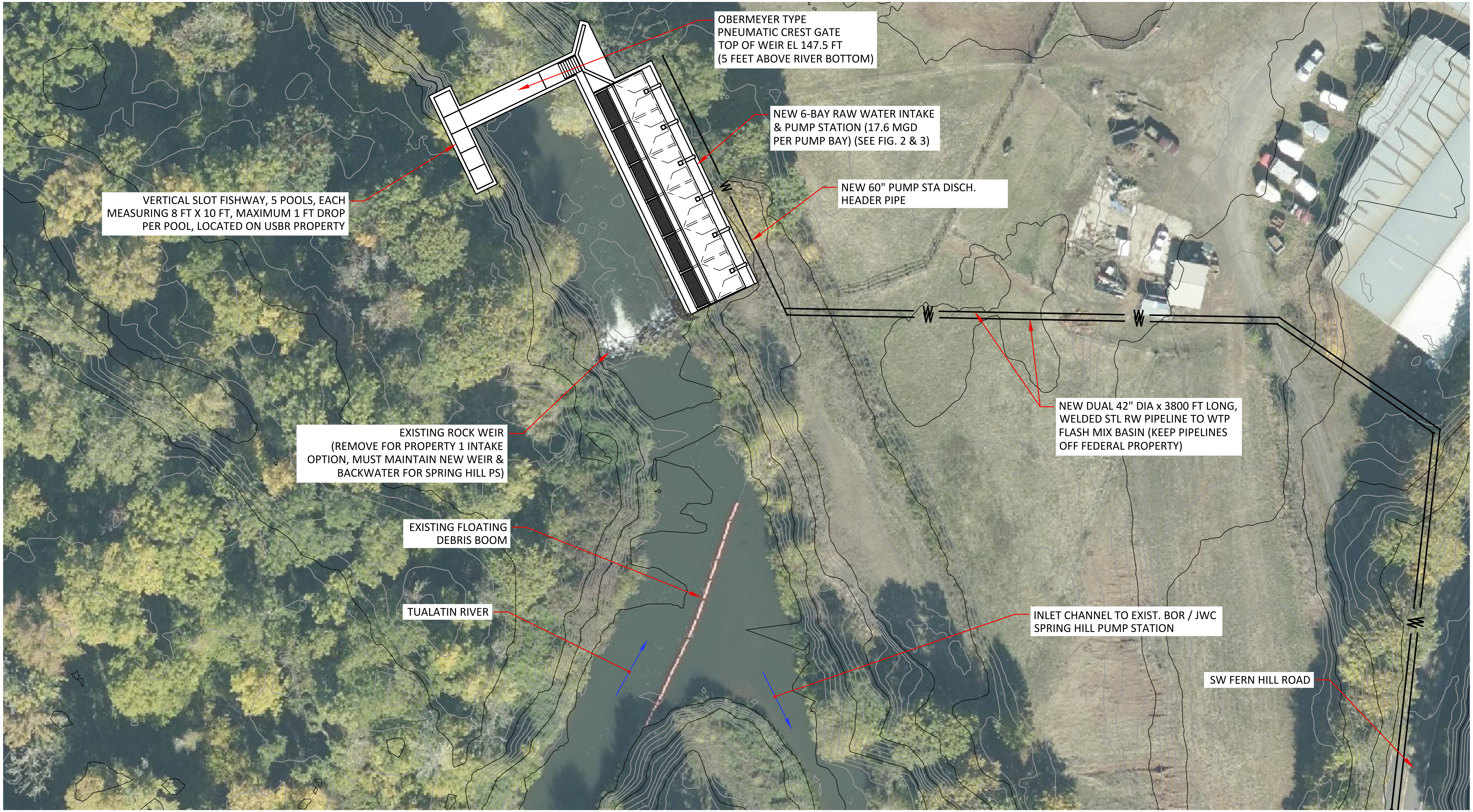
6.3.1 Intake Alternative Descriptions and Conceptual Layouts

The following sections review each intake alternative evaluated for the Tualatin River, including its design criteria and conceptual layout. The conceptual layouts correspond to a firm intake capacity of 88 mgd, which is JWC's initial firm intake capacity requirement. If the SHPP continues to be used in conjunction with a new intake until the end of its useful life, the new intake must have a minimum capacity of 44 mgd. However, all three of the options described in detail in the sections below consist of a firm and resilient intake capacity of 88 mgd.

6.3.1.1 Property 1: Develop a New, Seismically Resilient Intake on Property 1

The proposed intake on Property 1 borders the east bank of the Tualatin River and is located just downstream and to the north of the existing SHPP. In 2020, JWC hired Jacobs to prepare a concept layout and early analysis of a new intake facility located along this stretch of the Tualatin River. Figure 6.7 provides an overview site plan of the new intake facilities as conceptualized on this new property. Figures 6.8 and 6.9 present a concept plan view of and a typical section through the new intake, respectively.

This alternative follows the same intake style as developed in 2020 but modifies the geometry of the intake pump station's wet wells and screens to more closely and conservatively follow the *2018 National Oceanic and Atmospheric Administration (NOAA) Draft Salmonid Passage Design Guidelines (SPDG)* (NOAA Draft). The new intake concept will remove the existing rock weir, which provides backwater to the SHPP, and construct a new low-head concrete dam across the river to provide back-water elevation to both the new intake facility as well as to the SHPP. As part of the low-head concrete dam construction, a new fish ladder will be required to provide approximately five jumping pools with an assumed height of one foot per pool, for fish to travel around the new low-head dam. Replacement of the existing rock weir will require coordination with USBOR.


OBERMEYER TYPE
PNEUMATIC CREST GATE
TOP OF WEIR EL 147.5 FT
(5 FEET ABOVE RIVER BOTTOM)

NEW 6-BAY RAW WATER INTAKE
& PUMP STATION (17.6 MGD
PER PUMP BAY) (SEE FIG. 2 & 3)

NEW 60" PUMP STA DISCH.
HEADER PIPE

VERTICAL SLOT FISHWAY, 5 POOLS, EACH
MEASURING 8 FT X 10 FT, MAXIMUM 1 FT DROP
PER POOL, LOCATED ON USBR PROPERTY

NEW DUAL 42" DIA x 3800 FT LONG,
WELDED STL RW PIPELINE TO WTP
FLASH MIX BASIN (KEEP PIPELINES
OFF FEDERAL PROPERTY)

EXISTING ROCK WEIR
(REMOVE FOR PROPERTY 1 INTAKE
OPTION, MUST MAINTAIN NEW WEIR &
BACKWATER FOR SPRING HILL PS)

EXISTING FLOATING
DEBRIS BOOM

TUALATIN RIVER

INLET CHANNEL TO EXIST. BOR / JWC
SPRING HILL PUMP STATION

SW FERN HILL ROAD

N

PLAN

SCALE: 1"= 50'

0" 50' 100'

carollo

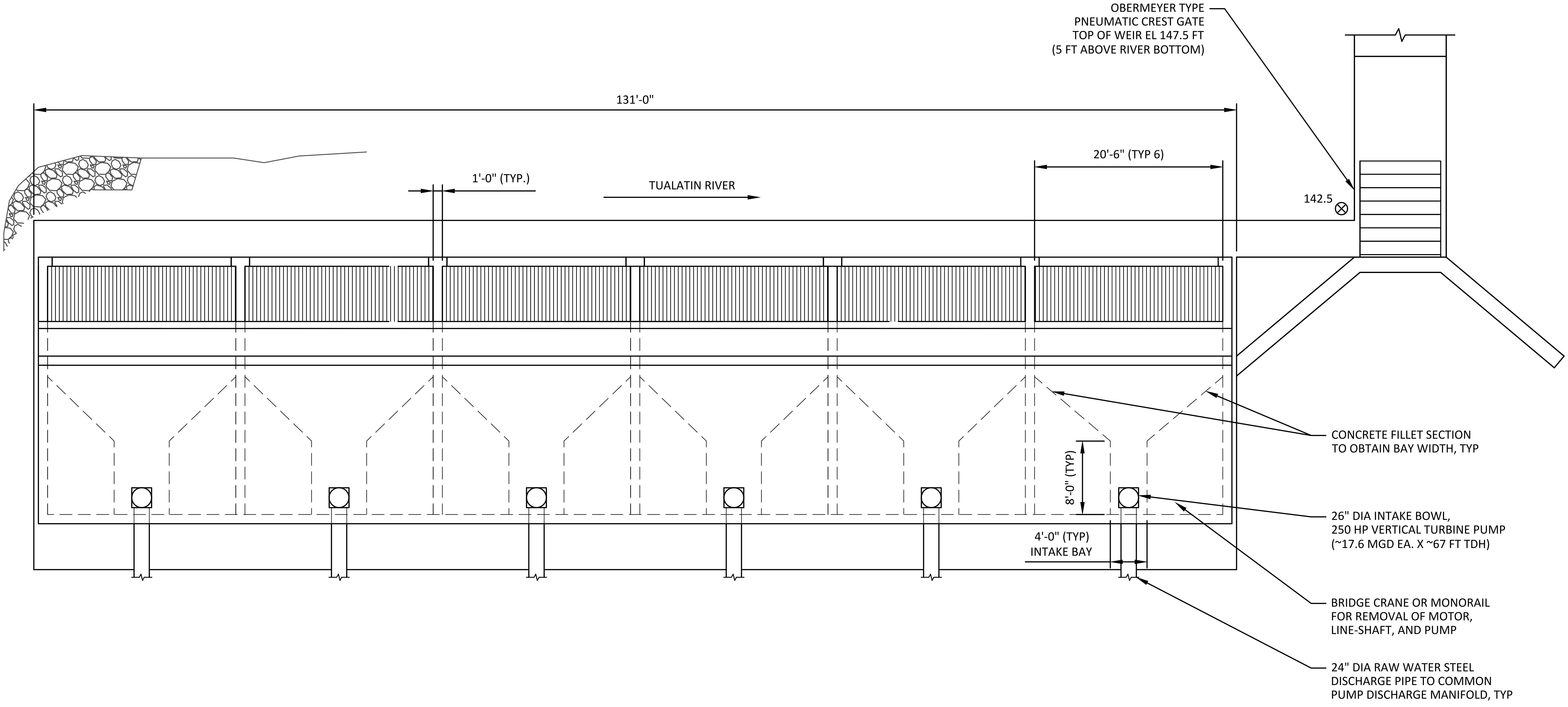
McMILLEN JACOBS ASSOCIATES

JOINT WATER COMMISSION

MASTER PLAN STUDY (TUALATIN RIVER INTAKE EVALUATION)

Figure 6.7
Property 1 Conceptual Intake Site Plan
(88-mgd Firm Capacity)

FEB. 2022



NOTE:

1. VERTICAL DATUM IS NAVD88.

PLAN

SCALE: 3/32"= 1'-0"

0' 8' 16'

carollo

McMILLEN
JACOBS
ASSOCIATES

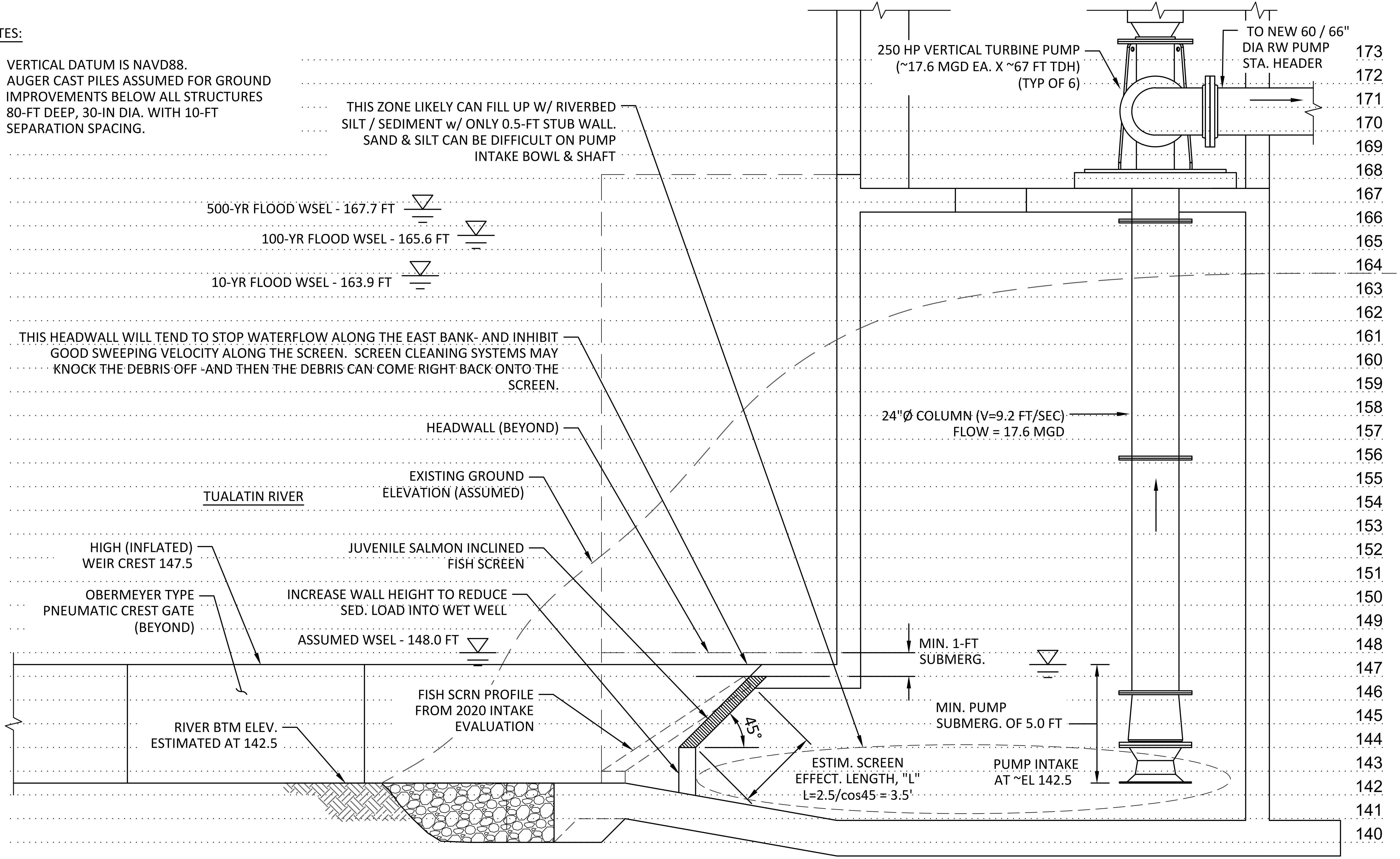
JOINT WATER COMMISSION
MASTER PLAN STUDY (TUALATIN RIVER INTAKE EVALUATION)

Figure 6.8
Property 1 Conceptual Intake and Pump Station Plan
(88-mgd Firm Capacity)

FEB. 2022

NOTES:

1. VERTICAL DATUM IS NAVD88.
2. AUGER CAST PILES ASSUMED FOR GROUND IMPROVEMENTS BELOW ALL STRUCTURES 80-FT DEEP, 30-IN DIA. WITH 10-FT SEPARATION SPACING.



SECTION

SCALE: 1/4"= 1'-0"

0' 4' 8'

carollo®

McMILLEN JACOBS ASSOCIATES

JOINT WATER COMMISSION

MASTER PLAN STUDY (TUALATIN RIVER INTAKE EVALUATION)

Figure 6.9
Property 1 Conceptual Intake and Pump Station Section
(88-mgd Firm Capacity)

FEB. 2022

The newly proposed intake on Property 1 is more than twice as large as that intake concept laid out in 2020 to meet the following design criteria:

- The clearance height between the riverbed, which has an assumed elevation of 142.5, and the invert of the inclined intake screens should be no less than 1.5 feet or preferably more to avoid siltation and solids deposition problems inside the new wet well structure. Previous layouts utilized only 0.5 feet for this clearance height.
- An appropriate screen angle of no less than 45-degrees off horizontal should be used to allow the screens to be effectively cleaned and remain in agreement with the NOAA Draft's SPDGs, as shown in Figure 6.9. Previous layouts utilized shallower screen angles of approximately 33-degrees off horizontal, which will have resulted in longer effective screen lengths but would be very difficult to clean and maintain.
- An effective screen height and length of no more than approximately 4 feet will be possible under shallow river conditions, even with the construction of a new low-head dam downstream of the intake site.
- Selection of an appropriate "design approach velocity" for active screens is a critical design parameter when sizing up new intake facilities. The NOAA Draft states:

> *The design approach velocity for active screens must not exceed 0.4 ft/s [feet per second] for fish screens where exposure time is limited to less than 60 seconds, or 0.33 ft/s, where exposure time is greater than 60 seconds.*

Note that, here, "exposure time" is defined as the time it takes a particle to traverse the length of the fish screen when moving at the speed of the sweeping velocity, which is essentially the stream velocity parallel to the screen face.

The first and second items of this list, when corrected, will push the overall screen system's length to over 100 feet in total for a new pump station with a firm capacity of 88 mgd. Given the shallowness and narrowness of the river at this site and the fact that a substantial downstream weir must be created to effectively back up water over the intake site, the system's sweeping velocity is almost assured to be low in the range of 1.0 ft/s. A sweeping velocity under 1.7 ft/s will result in an exposure time of greater than 60 seconds along the screen system. As a result, a design screen approach velocity of 0.33 ft/s is appropriate for this new intake site.

These four criteria demand that the width (W) for the intake's pump bays, designed to pass 17.6 mgd or 27.2 cubic feet per second (cfs), is approximately 20.5 feet:

- (W = 27.2 cfs / (0.33 ft/s * ~4-feet).

For a total of six pump bays of which five operate to provide a firm 88 mgd capacity, the overall length of the new intake will be approximately 131 feet, as shown in Figure 6.8. Each new pump will be designed to have a design flow capacity of approximately 17.6 mgd each and an estimated total dynamic head (TDH) of approximately 67 feet, which will result in vertical turbine pumps with vertical motors sized at 250 horsepower (hp) each. Such motors are in the safe and appropriate design range of 480-volt alternating current (VAC), 3-phase electrical power service.

Limitations and Potential Fixes

Given limited size and space along the bank of the Tualatin River at Property 1, the required increase in footprint is substantial. Such a large, or rather long, intake at this location, will likely be both expensive to build and difficult to maintain.

One possible fix to this alternative's shallow river design depth and long width of the new intake screens is to require the new downstream concrete weir and Obermeyer gate structure to have a minimum design height that is two to three feet higher than what was proposed in 2020. Raising this weir and gate structure will increase the water depth during low and normal flow periods, thus reducing the significant length of the screen system.

With that being said, raising the new weir structure would require longer and taller fish passage facilities, and may involve significant permitting implications associated with having a taller low-head dam on a river with ESA-listed fish.

New Raw Water Pipelines

JWC prefers two new raw water pipelines rather than one larger pipeline to convey flows from the new intake to the JWC WTP's rapid-mix basin for both redundancy as well as to maintain higher velocities that reduce settling of sediment in the pipe and enhance meter accuracy. For conveyance of a firm 88-mgd design flow, the new pipelines will likely be 42 inches in diameter and approximately 3,800 feet in length. The pipelines are routed towards SW Fern Hill Road's western shoulder to avoid the need to obtain new easements along federally owned lands that are adjacent to the east side of the existing inlet channel to the SHPP.

Seismic Considerations

Like the SHPP's existing site, potentially liquefiable soils are expected to extend 80 to 90 feet in depth at Property 1. To mitigate this seismic hazard, the new intake structure, gate, and other critical facilities must be supported on piles or ground-improvement columns. The auger-cast pile foundation system, which JWC is familiar with through its use at the JWC WTP, is one cost-effective approach.

To build in resiliency, the new raw water pipes are recommended to be constructed of welded steel with bell and spigot lap-welded joints or earthquake-resilient ductile iron piping (ERDIP).

Permitting Considerations

A new intake at Property 1 will entail work below OHW and result in JWC's additional withdrawal from the Tualatin River; therefore, this alternative will require permits from USACE, DSL, DEQ, ODWF, and NMFS. This alternative's location on the river, due to its shallow river depths, will require an intake structure that is larger than what is required for Property 2, given the need for more fish screen areas that meet ODFW's and NMFS's fish-screen criteria. The larger footprint will likely result in more notable consequences and effects on areas below OHW and within the floodplain.

ODFW and NMFS's fish-screen criteria also require this alternative to construct a low-head dam, which must include accommodations for fish passage. However, the placement of this permanent structure across the Tualatin River will make permitting challenging. The temporary consequences of construction activities, such as dewatering and fish-salvage operations, must also be considered.

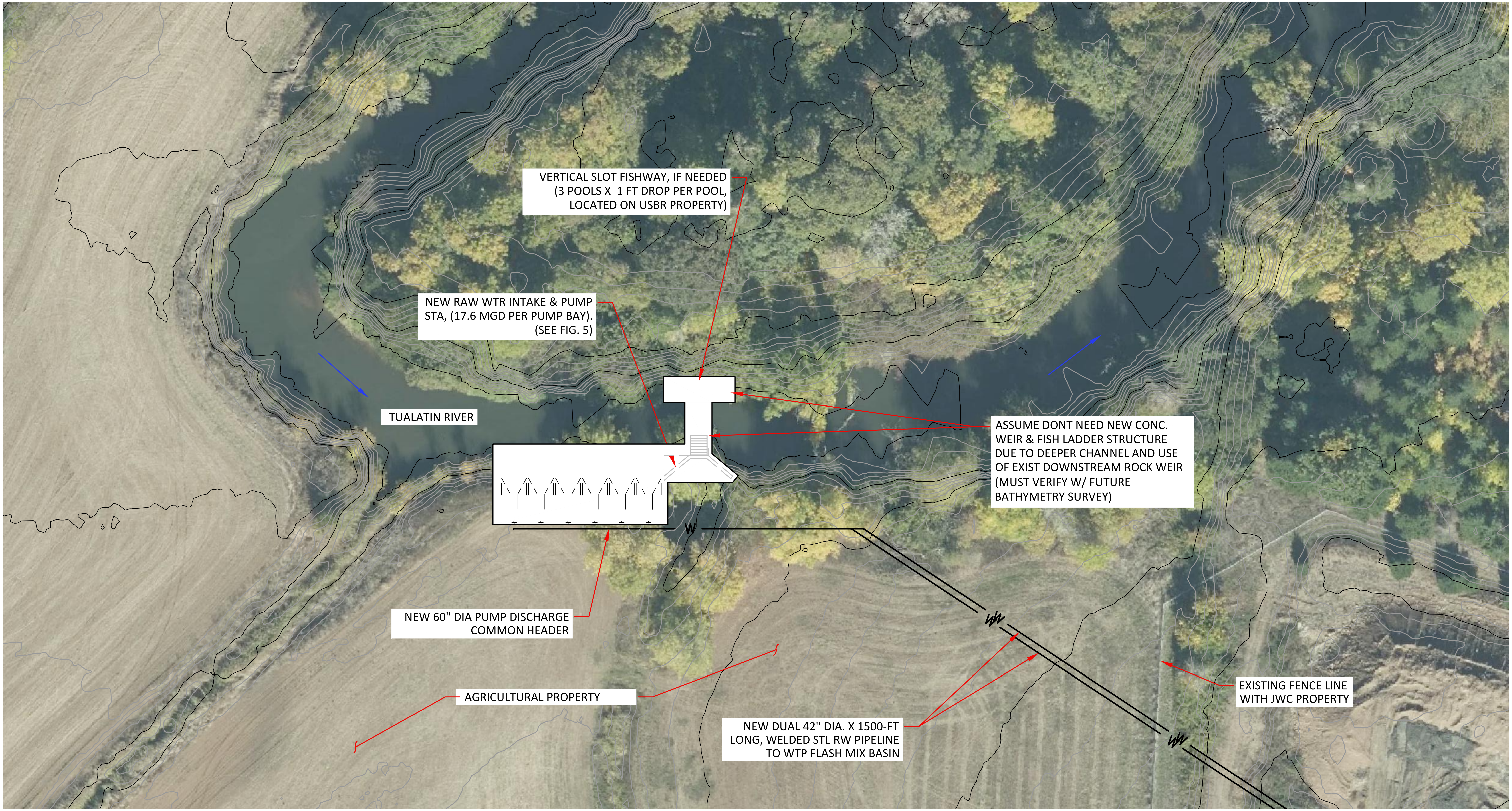
The new intake and dam are considered placement of fill within the floodplain and, if a net zero balance of cut and fill is not achievable, this alternative will require floodplain permitting through Washington County and compliance with FEMA guidelines. Note that the intake structure's placement within the floodplain will be considered an additional fill and increase the challenge of balancing cut and fill on the property.

Finally, in addition to the new intake facility itself, this alternative will install approximately 3,800 feet of a raw water pipeline to convey the water from the new intake to the JWC WTP. This pipeline's proposed alignment must be evaluated for the presence of wetlands and, if impacts are unavoidable, they must be addressed through permitting and mitigation.

6.3.1.2 Property 2: Develop a New, Seismically Resilient Intake on Property 2

The proposed intake site on Property 2 is part of a larger agricultural property that shares a boundary along the JWC WTP's western edge and is located approximately 800 feet upstream of the existing rock weir.

Figure 6.10 presents an overview site plan of the new intake facilities as conceptualized on this property. Figures 6.11 and 6.12 present a concept plan view of and a typical section through the new intake, respectively. Note that, to allow for like comparisons of the two alternatives, Property 2's conceptual layout follows the same intake style as developed for Property 1.



N

PLAN

SCALE: 1"= 50'

0" 50' 100'

carollo®

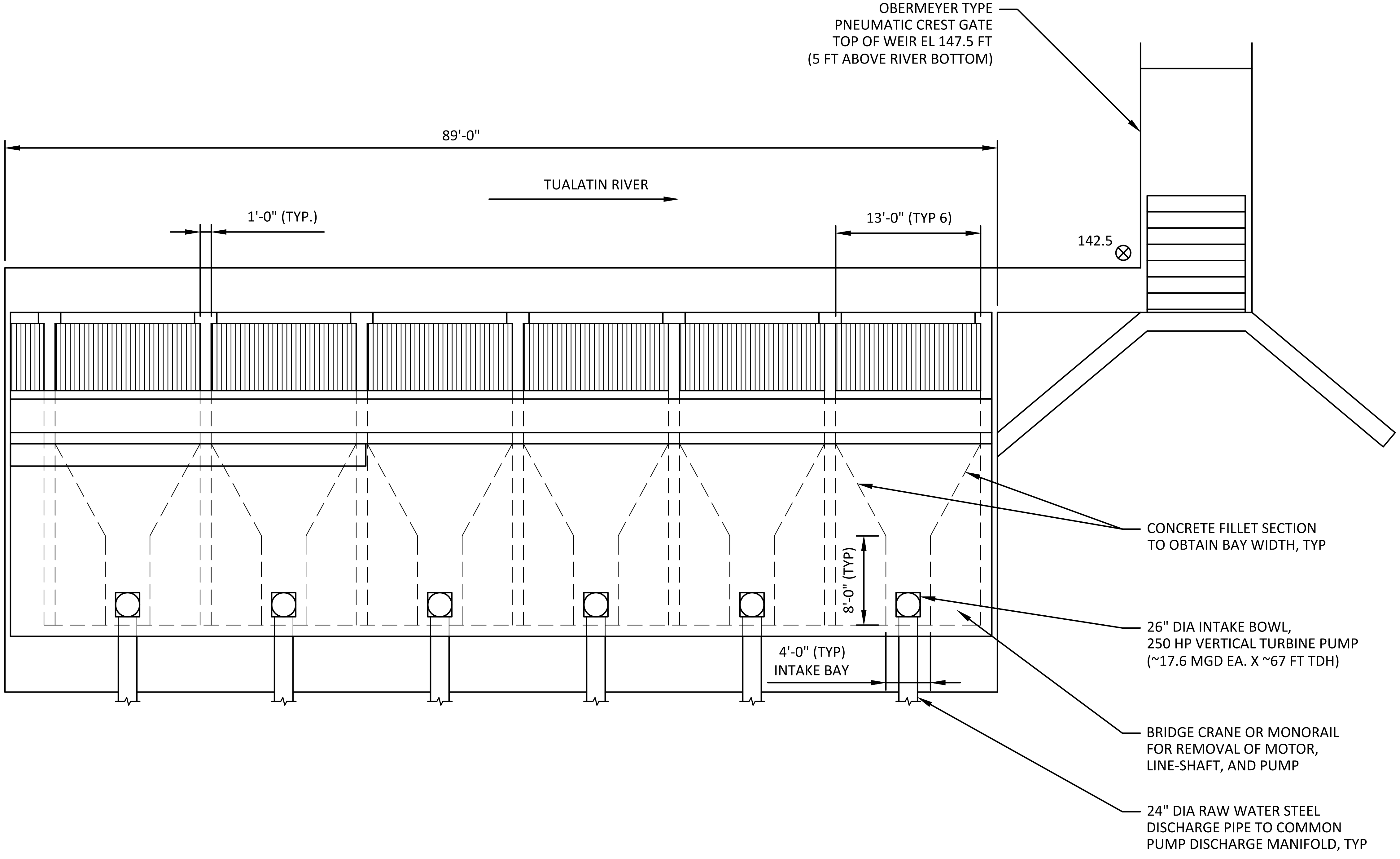
McMILLEN JACOBS ASSOCIATES

JOINT WATER COMMISSION

MASTER PLAN STUDY (TUALATIN RIVER INTAKE EVALUATION)

Figure 6.10
Property 2 Conceptual Intake Site Plan
(88-mgd Firm Capacity)

FEB. 2022



NOTE:

1. VERTICAL DATUM IS NAVD88.

N

PLAN

SCALE: 3/32"= 1'-0"

0' 8' 16'

carollo

McMILLEN JACOBS ASSOCIATES

JOINT WATER COMMISSION

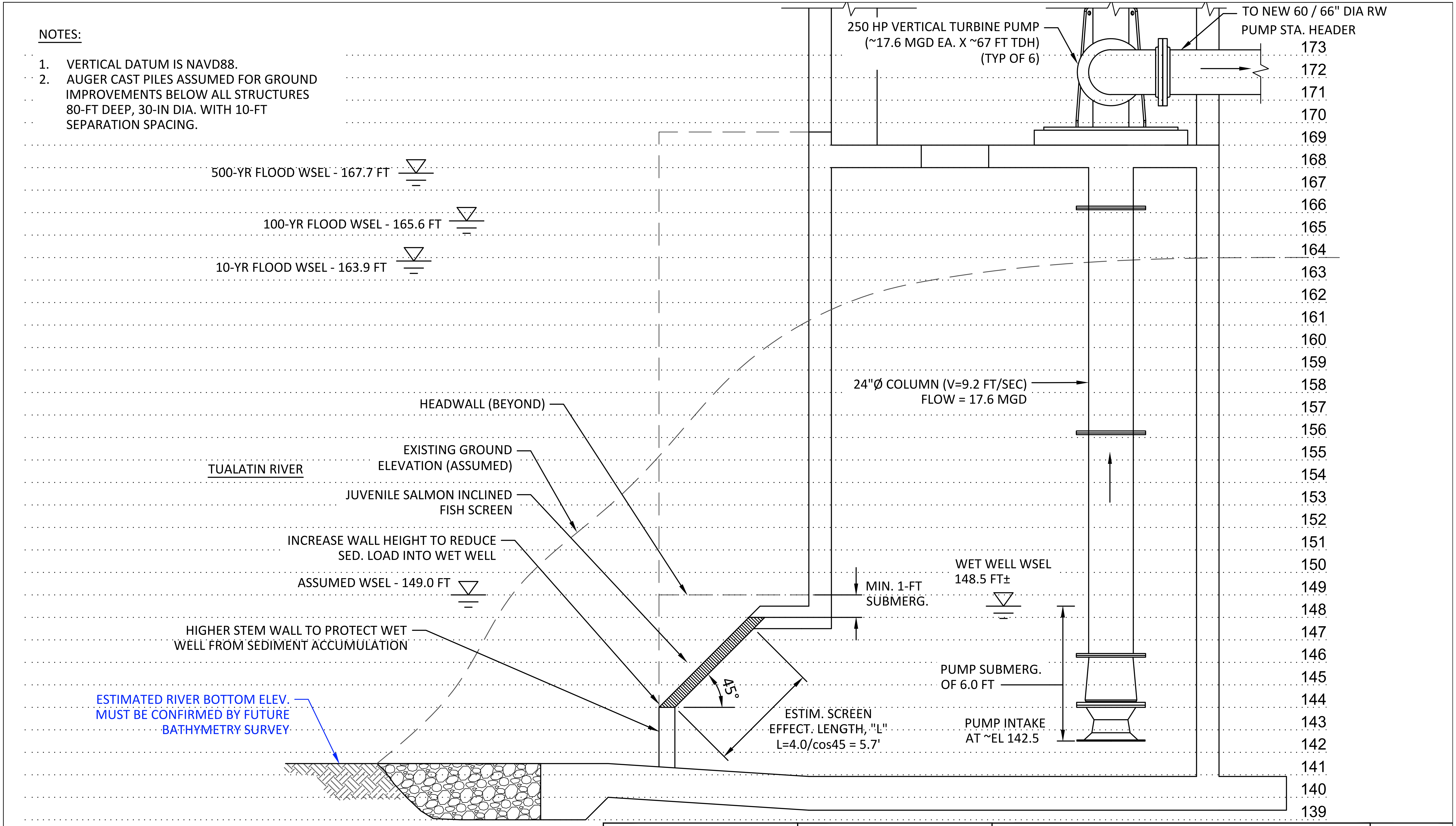
MASTER PLAN STUDY (TUALATIN RIVER INTAKE EVALUATION)

Figure 6.11
Property 2 Conceptual Intake and Pump Station Plan
(88-mgd Firm Capacity)

FEB. 2022

NOTES:

1. VERTICAL DATUM IS NAVD88.
2. AUGER CAST PILES ASSUMED FOR GROUND IMPROVEMENTS BELOW ALL STRUCTURES 80-FT DEEP, 30-IN DIA. WITH 10-FT SEPARATION SPACING.



SECTION

SCALE: 1/4"= 1'-0"

0' 4' 8'

carollo

McMILLEN JACOBS ASSOCIATES

JOINT WATER COMMISSION
MASTER PLAN STUDY (TUALATIN RIVER INTAKE EVALUATION)

Figure 6.12
Property 2 Conceptual Intake and Pump Station Section
(88-mgd Firm Capacity)

AUG. 2022

Data from an official bathymetry survey is currently unavailable to map the river bottom at Property 2; as such, further development of this alternative's layout will require a professional bathymetric survey that confirms concept design assumptions. With that being said, the Carollo team's visit to the site and basic observations indicate that the river is slightly narrower and running at deeper water depths with higher sweeping velocities than what is observed at Property 1.

This result is partly due to Property 2's location upstream of the rock weir, which deepens the water depth in the narrowed channel, and along the south bank of the Tualatin River on an outswing curve; the resulting high sweeping velocities along the new intake screens will be beneficial for screen cleaning. This river depth also eliminates the need to construct a new low-head concrete cut-off dam and fish ladder. Final determinations of this property's needs will follow the formal bathymetry survey and as the preliminary design layout is finalized.

As introduced earlier, Figure 6.12 shows a concept estimate of a section through the river bottom and intake screen area with an assumed river water surface elevation of 149.0 and an assumed river bottom elevation between 141.0 and 142.0. At these elevations and providing a conservative 2.5-foot-tall stem and cutoff wall below the intake screens, an effective screen design length and height of roughly 5.3 feet is estimated. Given the higher sweeping velocities anticipated at this site, relative to Property 1, a standard 0.4-ft/s design approach velocity is assumed for the new screen system.

Each pump bay will be designed to pass 17.6 mgd (i.e., 27.2 cfs), necessitating a screen width of approximately 13 feet:

- (W = 27.2 cfs / [0.40 ft/s * 5.3 ft])

This screen width is substantially shorter than the 20.5-foot screen width estimated for the new intake at Property 1 and, thus, represents the potential for significant savings in construction costs.

For a total of six pump bays of which five operate to provide a firm 88-mgd capacity, the overall length of the new intake will be approximately 90 feet, as shown in Figure 6.11. Each new pump will be designed to have a design flow capacity of approximately 17.6 mgd each and an estimated TDH of approximately 67 feet, which will require vertical turbine pumps with vertical motors sized at 250 hp each. As noted earlier, such motors are in the safe and appropriate design range of 480-VAC, 3-phase electrical power service.

New Raw Water Pipelines.

As with Property 1, JWC prefers two new raw water pipelines rather than one larger pipeline to convey flows from Property 2's new intake site to the JWC WTP's rapid-mix basin. To convey a firm design flow of 88 mgd, the new pipelines will be 42 inches in diameter and approximately 1,500 feet in length. The pipelines are assumed to be routed along the north side of the WTP site and to either the existing or a newly modified rapid-mix basin.

Seismic Considerations

The seismic considerations for Property 2 are the same as those for Property 1. An auger-cast pile foundation system is recommended to mitigate this site's seismic hazard from potentially liquefiable soils. For the new raw water pipes, welded steel or ERDIP is recommended.

Permitting Considerations

The permitting associated with the construction and operation of a new intake at Property 2 is similar to that of Property 1, aside from the following notable exceptions:

- The assumed deeper river depth (to be confirmed) along this reach allows for a smaller intake footprint required to meet fish-screening guidelines. This will allow for fewer impacts below OHW and less fill within the floodway than what is required for Property 1.
- Placement of an intake at this location may not require a low-head dam across the Tualatin River, which will greatly simplify permitting.
- This intake location will require a much shorter raw water pipeline and, therefore, less potential for impacts on wetlands.

6.3.1.3 Intake Alternative 3: Upgrade Existing Spring Hill Pumping Plant

The existing SHPP does not meet JWC's supply needs for two main reasons:

- The SHPP's firm capacity of 67 mgd is less than the required 88-mgd firm capacity in the near term.
- The SHPP and existing rock weir are not seismically resilient and, therefore, do not provide a seismically resilient supply of 44 mgd.

Note that this master plan did not evaluate the SHPP's capacity or seismic upgrades to it. Instead, the discussion and recommendations summarized herein are founded on previous studies and discussions with JWC staff.

Capacity Considerations

The SHPP currently has a total capacity of 88 mgd and a firm capacity of approximately 67 mgd. The capacity of the SHPP must be increased if JWC wishes to rely solely on this intake to meet its future water demands.

Because the SHPP and existing rock weir are owned by USBOR, JWC cannot make any modifications to these structures without receiving approval from this federal agency. In the past, even minor modifications have not been approved by USBOR. As such, getting approval from USBOR to make modifications to the intake structure or weir is anticipated to be very difficult. Because the Tualatin Valley Irrigation District also uses the SHPP and rock weir, any modifications would need to be coordinated with them as well.

JWC has considered the following options to expand their capacity through the SHPP:

- **Attempt to lease one additional pump pad and wet well location from TVID and install a new JWC pump**:
  - This option was evaluated during the development of the JWC WTP's facility plan and found to be cost-prohibitive.
- **Use an intertie to the TVID pipeline at either the SHPP or JWC WTP**:
  - A pressure-regulating valve will be required to implement this option, because TVID's pumps operate at approximately 160 pounds per square inch (psi) while JWC requires just 30 psi. Furthermore, JWC's use of any TVID infrastructure will create legal complications since TVID pays discounted power rates while JWC is required to pay regular rates.

Seismic Considerations

Because the SHPP and rock weir are owned by USBOR and used by TVID and JWC, the cost of any seismic upgrades should be shared by these entities. However, seismically upgrading the SHPP is not a priority for TVID or USBOR, from whom getting approval for the modifications will be required. As such, JWC may be required to bear the cost of the upgrades alone.

Upgrades to secure the SHPP's equipment were performed as part of the 85-mgd WTP Expansion Project. However, all other SHPP seismic upgrades that were recommended in previous studies have not been implemented. Today, seismically upgrading the SHPP will consist of ground improvements, structural upgrades, and improvements to or replacement of the raw water pipeline.

The most feasible ground-improvement method will likely be jet-grouting through the base of SHPP to mitigate risks of liquefaction and lateral spreading. According to geotechnical information from the nearby WTP, depths of the liquefiable soils (predominantly fine-grained silt and sandy silt deposits) are likely to be on the order of 80 to 90 feet. Considering the extent of the jet-grouting required and difficulties associated with drilling through existing foundation slabs, the ground-improvement cost will be approximately $8.0 million.

Carollo performed an in-depth seismic structural evaluation of the SHPP and documented results in the *2008 Seismic Mitigation Study*. Table 6.1 lists the seismic-mitigation concepts proposed by that study, their costs in 2008 dollars, and their costs in 2022 dollars escalated using Engineering News-Record's 20-city Construction Cost Index.

Table 6.1 SHPP Seismic Mitigation Concepts from 2008 Seismic Mitigation Study

Proposed Mitigation Concept	Cost (2008 Dollars)	Cost (2022 Dollars)
Add concrete shear walls at the lower level along the north, east, and west sides. Dowel wall-reinforcing into the existing concrete floor slab above and mat foundation at the base.	$2,065,000	$3,240,000
Add steel tie plates with epoxy anchors at the top of and between the double tee flanges to create a continuous roof diaphragm.	$200,000	$310,000
Add a continuous steel angle at the perimeter of the roof with epoxy anchors between walls and the top of the double tee flanges.	$80,000	$130,000
Add a second flexible joint in each pump discharge line.	$47,000	$70,000
Total	**$2,392,000**	**$3,750,000**

At least one of the two existing raw water pipelines must be improved or replaced to meet JWC's seismic LOS goals. The transmission pipelines must also be upsized or paralleled to achieve the interim and future expansion capacities if the full supply is to be pumped from the SHPP.

Replacing the 36-inch raw water pipeline with a new resilient 42-inch pipeline along the same pipeline alignment is estimated to have a construction cost of approximately $5.0 million, assuming an overall pipeline length of 3,000 feet and an installed unit cost of $40 per inch diameter per linear foot. For a 42-inch-diameter steel pipe, the unit-installed cost is assumed to be $1,680 per linear foot.

Permitting Considerations

Currently, the SHPP fish screens do not meet ODFW or NMFS fish screen criteria and, since the SHPP is owned by USBOR, any modification to the SHPP requires USBOR authorization. Triggered by conditions on two water right permits and one pending water right application for JWC's use at SHPP, JWC and ODFW worked together to develop a mitigation plan to meet ODFW's fish protection requirements (ORS 498.316) at the SHPP fish screens. In 2019 JWC assessed the annual potential entrainment rate of juvenile salmonids into the SHPP, and the results of this study allowed JWC and ODFW to develop a mitigation plan. Bateman Creek was selected as a preferred mitigation site based on its location within the watershed, presence of priority fish passage barriers, and the valuable cold-water habitat it provides. At the time of this document's writing, JWC has begun construction of the Bateman Creek fish passage restoration project and, when completed, will satisfy the mitigation required for ODFW's 2020 fish screening exemption for JWC's pumps at the SHPP.

There is the potential that in-water work at SHPP and the necessary state authorization(s) could trigger a subsequent ODFW review of the fish screens; however, the Bateman mitigation agreement requires that "the Applicant maintain their existing screens to a level that is equal to or exceeding their current functionality and fish protection ability." Therefore, as long as the SHPP screens remain as functional as they are today, the fish screening exemption should cover the modifications required to facilitate future JWC operations at the SHPP. This would need to be confirmed with ODFW prior to SHPP modifications.

Should it become necessary, meeting NMFS and ODFW fish screening criteria at SHPP would require either replacing the existing rock weir with a new concrete dam that has a higher crest elevation and/or attempting to widen the existing intake screens. The latter option is likely not viable due to the real-estate constraints on either end of the existing structure and the geometry of the current wet well's pump bays being virtually fixed. Therefore, if meeting fish screening criteria is required, a new low-head concrete dam would likely be required.

Since a dam across the Tualatin River would have fish passage implications of its own, JWC would likely be encouraged to enter into a new agreement with ODFW to mitigate fish passage impacts with habitat restoration, thereby providing a greater benefit to native and ESA-listed fish than the potential impacts of constructing a dam across the Tualatin.

The SHPP structural upgrades required to meet JWC's long-term demands and seismic resiliency goals could trigger the need for environmental and land use permits. Any in-water work associated with SHPP upgrades will require authorization from USACE, DSL, DEQ, ODFW, NMFS, CWS, and Washington County and can take up to two years to permit, given potential short- and long-term impacts on ESA-listed and native fish, to regulated wetlands and water, and within the floodplain. The in-water work will require ODFW approval of a Fish Passage Plan and, as mentioned for Property 1's permitting considerations, temporary consequences of construction activities, such as dewatering and fish-salvage operations, must also be considered.

Operations of USBOR's Tualatin Project, which includes the SHPP, is governed in part by NMFS' 2014 Biological Opinion. The approved water withdrawal in this Biological Opinion is based on "…the total quantity of water that will be pumped through the Spring Hill pumping plant during the period of time that juvenile steelhead are most likely to be in proximity to the intake screens – March through June—of 30,000 acre-feet…". This cumulative, March through June, withdrawal reflects combined JWC and TVID use. The combined volume of TVID and JWC pumping reported at SHPP during these months has fluctuated from 11,310 acre-feet to 18,576 acre-feet since 2012, with higher demands occurring in years with dry or warm spring weather (e.g., 2015, 2021). There is currently no agreement in place regarding the individual usage of SHPP from March through June between JWC and TVID.

Future water demands from March through June incorporate forecast demands during both the non-peak and peak demands seasons. Based on demand forecasts in Section 5.4.2.1 and 5.4.2.2, 2040 average day demand combined with injection associated with JWC and JWC partner aquifer storage and recovery (ASR) projects are forecast to exceed 30,000 acre-feet by 2040. Because these forecast demands are sensitive to the timing of ASR injection and treatment plant capacity expansion, the JWC should re-evaluate demand projections with each master plan update to ensure that combined TVID and JWC water use continues to be below the limit of 30,000 acre-feet from March through June. Re-initiation of a formal ESA Section 7 consultation will be required if JWC proposes increases to water use beyond this amount at the SHPP.

6.3.2 Tualatin River Intake Economic Evaluation

Comparative cost estimates were prepared to evaluate the economic impacts of each intake alternative. The estimates for alternatives 1, 2a, and 2b are equivalent to an Association for the Advancement of Cost Engineering (AACE) Class 5 estimate, used for "order-of-magnitude" cost comparison, with an expected accuracy range between minus 20 percent to minus 50 percent on the low end and plus 30 to plus 100 percent on the high end. All costs presented are in February 2022 dollars.

Direct construction capital cost estimates focused on major facilities and equipment required with additional allowances for mechanical and electrical requirements. Total construction cost estimates also include the following percentage factors that were applied to direct costs:

- **General conditions**: 12 percent.
- **Contractor overhead and profit**: 10 percent.
- **Engineering, administration, and permitting:** 25 percent.

Note that these costs are not adjusted to consider future inflation rates.

Table 6.2 presents the costs to construct an intake on Property 1 and Property 2. As can be seen, two different conceptual-level cost estimates were prepared for Property 2: One assumes that a new low-level, concrete cut-off dam and fish ladder structure are required while the second assumes that neither is required. Once again, the final determination of the intake’s needs will follow the official bathymetric river survey conducted at the site.

The total estimated cost of building an intake on Property 1 is $49 million. If a concrete weir and fish ladder are not required, Property 2’s cost is estimated to be approximately half that of constructing on Property 1 at $25 million. If a weir and fish ladder are required, Property 2’s cost will be approximately $34 million, still significantly less than building an intake on Property 1.

Table 6.3 presents an AACE Class 10 cost estimate to upgrade the SHPP, which includes a 30 percent scope contingency since the approach to increasing this intake’s capacity is not well-defined at this time. Upgrades are estimated to cost $48 million, although this cost should not be directly compared to those of the other alternatives since this project is not as well defined as those that build new intakes.

Table 6.2 Conceptual-Level Cost Comparison of Tualatin River Intake Alternatives

No.	Alternative Name	New Mixed Flow Pumps (250 hp ea w/VFDs) and Mech Piping ($M)	Cofferdam / Temp. Const. Works for Intake W-Well ($M)	Intake and PS Struct, Mech, Ele, I&C Construction ($M)	Civil Grade Build-up / Retaining Walls & Bank Rip-Rap ($M)	2 of 42 in Dia. RW Transmission Pipeline ($M)	RW Transmission Pipe X-ing Exist Yard Pipe ($M)	Cofferdam / Temp. Works for Conc. Weir and Fish Ladder ($M)	Conc. Weir and Obermeyer Gate Structure ($M)	Fish Ladder Structure ($M)	Construction Subtotal ($M)	General Conditions and Contractor OH&P ($M)	Engineering, Admin., and Permitting	Estimated Concept Project Cost ($M)
1	Property 1: • 88-mgd intake and pump station. • Concrete weir structure w/ gates. • Fish ladder (five-steps). • 3500-ft x 42-inch-diameter RWPs.	$2.2M (6 pumps and mech piping at $360K ea)	$1.5M	$8M (for 130-ft-long structure)	$2M	$10.2M (3800 LF pipe: $32/in-ft x 42 in x 2 x 3800 LF)	$0.3M (assume two crossings at $150K ea)	$1.5M	$4.5M	$1.2M	$31.4M	$7M	$10M	$49M
2A	Property 2: • 88-mgd intake and pump station. • Concrete weir structure w/ gates. • Fish Ladder (three-steps). • 1700-ft x 42-inch-diameter RWPs.	$2.2M (6 pumps and mech piping at $360K ea)	$1.5M	$6M (for 90-ft-long structure)	$1.6M	$4.6M (1700 LF pipe: $32/in-ft x 42" x 2 x 1700 LF)	$0.15M (assume one crossing at $150K ea)	$1.5M	$3.7M	$0.8M	$22.0M	$5M	$7M	$34M
2B	Property 2: • 88-mgd intake and pump station. • No concrete weir structure. • No fish ladder. • 1700-ft x 42-inch-diameter RWPs.	$2.2M (6 pumps and mech piping at $360K ea)	$1.5M	$6M (for 90-ft-long structure)	$1.6M	$4.6M (1700 LF pipe: $32/in-ft x 42 in x 2 x 1700 LF)	$0.15M (assume one crossing at $150K ea)	0	0	0	$16.0M	$4M	$5M	$25M

Notes:

Abbreviations: $K – thousand dollars; $M - million dollars; ea - each; Ele - electrical; ft - feet; I&C - instrumentation and control; LF - linear feet; Mech - mechanical; OH&P - overhead and profit; PS - pump station; RW - raw water; RWP - raw water pipeline; Struct - structural; VFD - variable frequency drive.

Table 6.3 Cost Estimate to Upgrade SHPP

No.	Alternative Name	Ground Improvement Seismic Mitigation ($M)	Structural Seismic Mitigation ($M)	42 in Dia. RW Transmission Pipeline ($M)	Cofferdam / Temp. Works for Conc. Weir & Fish Ladder ($M)	Conc. Weir ($M)	Fish Ladder Structure ($M)	30% Scope Contingency for Capacity Upgrades ($M)	Construction Subtotal ($M)	General Conditions and Contractor OH&P ($M)	Engineering, Admin., and Permitting ($M)	Estimated Concept Project Cost ($M)
3	Upgrade SHPP: 88 mgd	$8.0	$3.8	$5.0	$1.5	$4.5	$1.2	$7.2	$31	$7	$10	$48

6.3.3 Tualatin River Intake's Non-Economic Evaluation

In addition to the economic analysis, a non-economic analysis was used to further evaluate the three intake alternatives. Non-economic criteria were developed with input from JWC WTP staff and JWC partners who, through a collaborative process, assigned relative weights to each criterion to reflect JWC's priorities, with weights ranging from 1 (lowest importance) to 5 (highest importance).

Table 6.4 summarizes the non-economic criteria and weightings developed for this analysis.

Table 6.4 Non-Economic Criteria and Weighting

Criteria	Weight	Description
Water quantity	5	• Provides 44 mgd of seismically resilient supply. • Provides 88 mgd of initial supply. • Provides 105 mgd of interim supply. • Provides 125 mgd of build-out supply. • Diverts all JWC's existing water rights.
Resilience and reliability	4	• Intake location minimizes vulnerability to changing river morphology and sediment-transport issues. • Natural river geology, stream geometry, and stream hydraulics provide an intake site that keeps screens submerged and improves the reliability of JWC's water supply. • Site allows for easy, reliable design to get electrical equipment out of floodplains. • Intake is resilient to emergency disruptions such as earthquakes and flooding.
Land acquisition	2	• Simplifies land acquisition. • Minimizes need for new easements for raw water transmission piping.
Environmental / land use	4	• Minimizes environmental effects on waterways (including the existing SHPP intake), wetlands, soil, air, and sensitive habitat during construction and operation. • Minimizes effects on ESA-listed or sensitive species, fish, and other wildlife during construction and operation. • Minimizes requirements for permit acquisitions. • Minimizes likelihood of uncovering cultural resources.
Constructability	3	• Site provides for easy provision of 3-phase, 12.47kV power with space for transformers to provide 480-VAC electrical power for pumping. • Minimizes the construction schedule and risk. • Maximizes energy efficiency and efficient use of other resources during construction. • Minimizes need for new easements for raw water transmission piping.
O&M	3	• Ease of operation and maintenance. • Site provides for easy operations access to the intake on new gravel roadways that are not susceptible to flooding. • Maximizes energy efficiency (i.e., allows for use of high-efficiency mixed-flow vertical pumps) and efficient use of other resources during operation.

Notes:
Abbreviations: kW - kilowatt; O&M - operations and maintenance.

Each alternative was evaluated and scored on a scale of 1 (worst in meeting criterion) to 5 (best in meeting criterion). To determine a score per criterion and weighted overall score per alternative, assigned scores were multiplied by each criterion's corresponding weight factor.

Figure 6.13 summarizes the results of the non-economic evaluation. Appendix K breaks down the scoring and associated reasoning.

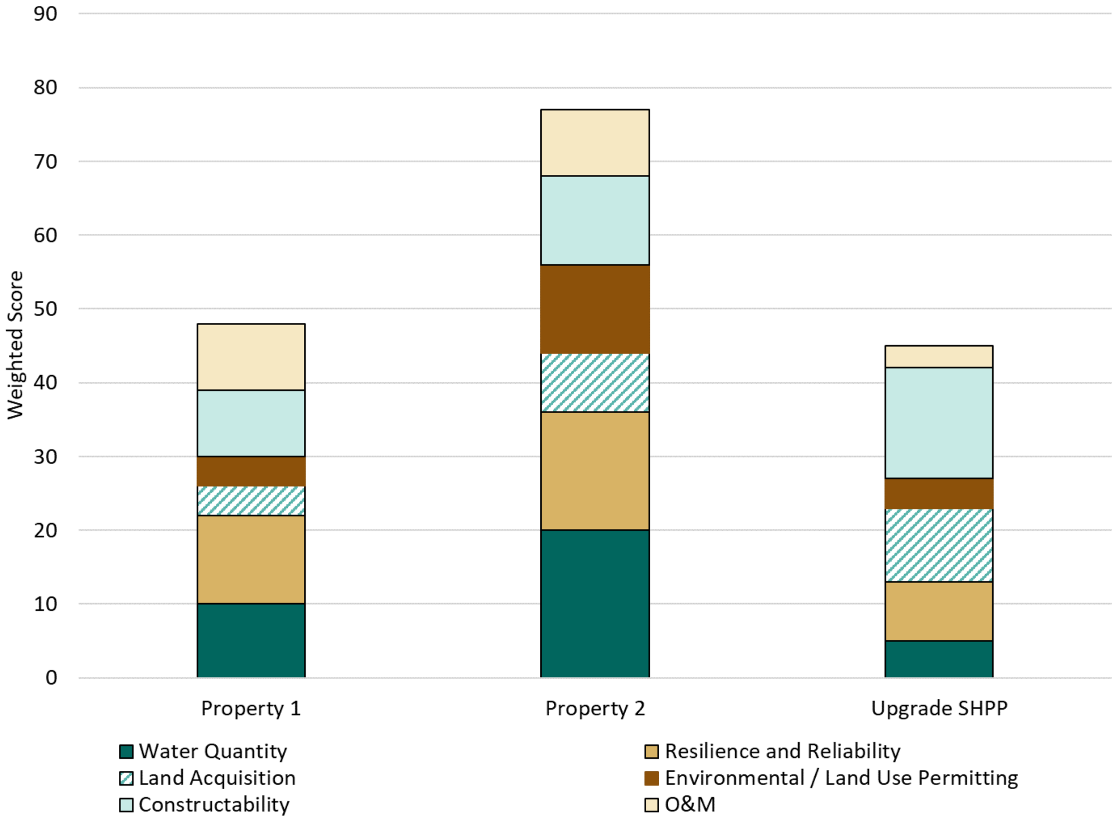


Figure 6.13 Intake Alternatives Non-Economic Scores

As shown in Figure 6.13, Property 2 scores the highest, meeting the most important criteria of water quantity, resilience and reliability, and environmental/land use. The depth of the river at Property 2 allows JWC to meet its intake capacity goals with the smallest footprint and, because it requires the shortest raw water pipeline, Property 2 is also considered the most resilient. In addition, Property 2 scores the highest for environmental permitting and land use because it has a small intake footprint and likely does not require a concrete weir and fish ladder, thus minimizing environmental impacts and, in turn, considerably simplifying permitting needs. Overall, JWC will have more control over the resilience and reliability of this new intake since they will own the property and intake structure.

Upgrading the SHPP scores the lowest, notably in water quantity and environmental/land use, because the approach to increasing the intake's capacity and subsequent coordination with USBOR and permitting is not predictable at this time.

6.3.4 Recommendations

The best long-term solution for JWC is to construct and own an intake facility on Property 2. A real estate professional may assist in negotiating a lot line adjustment with the owners of Property 2 to append approximately six additional acres to the JWC WTP's property.

If acquiring the proposed site on Property 2 is infeasible or if constructing a new intake is cost-prohibitive, JWC is recommended to further evaluate the option to upgrade the SHPP.

6.4 Evaluation of a Raw Water Pipeline from Scoggins Dam

When Scoggins Dam was constructed in the mid-1970s, an outlet for a municipal and industrial raw water pipeline was installed. Although funding constraints prevented such a pipeline from ever being built, in the early 2000s, JWC, TVID, and Clean Water Services (CWS) considered jointly building a raw water pipeline from Scoggins Dam to convey stored water in Hagg Lake.

Figure 6.14 shows the 6.5-mile route for the raw water pipeline, as proposed in the Murraysmith and CH2M Hill 2004 report *Raw Water Pipeline Routing Analysis and Preliminary Environmental Review Report* (2004 Report) and indicates easements JWC and CWS acquired between 2007 and 2010. At this time, approximately 48 percent of the route is either owned by JWC and CWS or has an easement for the raw water pipeline. The properties owned by JWC and CWS have specific easements for the raw water pipeline route due to permanent conservation easements with the U.S. Department of Agriculture Natural Resources Conservation Service (NRCS).

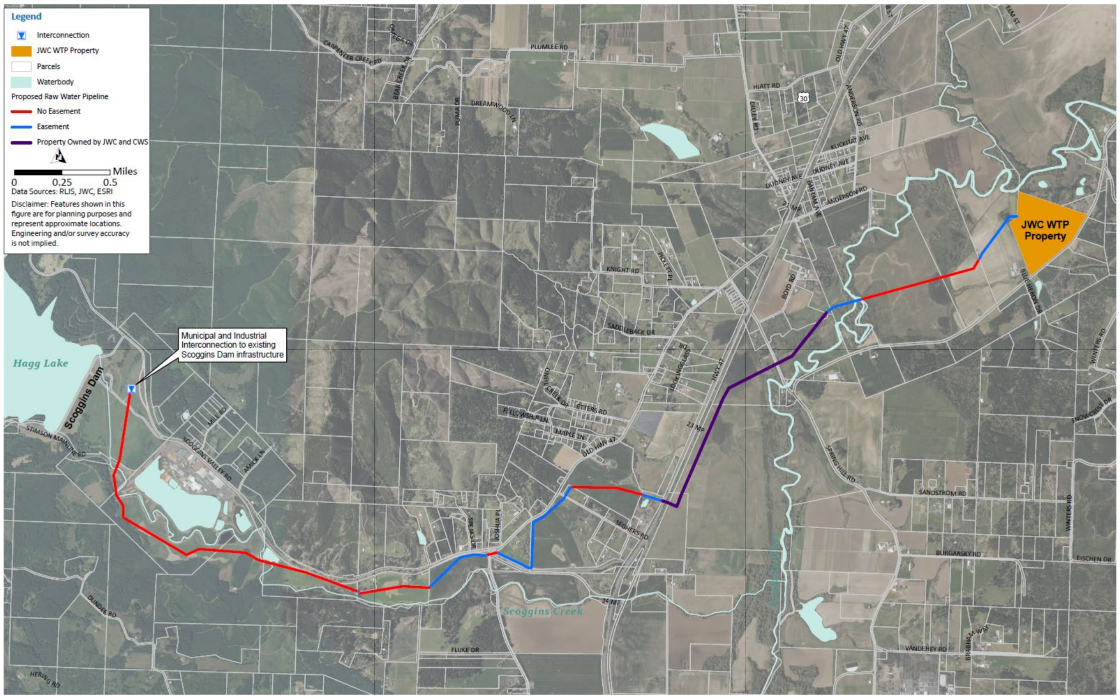


Figure 6.14 Proposed Raw Water Pipeline Route and Easements

This section evaluates the benefits and challenges associated with constructing a raw water pipeline from Scoggins Dam/Hagg Lake to the JWC WTP to supplement JWC's raw water supply from the Tualatin River.

6.4.1 History of the Project

While a municipal and industrial raw water pipeline was never built from Scoggins Dam, USBOR constructed the SHPP to supply the newly formed TVID and new JWC WTP. The Tualatin River channel conveys both JWC's natural flow water rights and stored water released from Hagg Lake and Barney Reservoir to the SHPP. These water rights are further discussed in Section 7.4.2.

In the early 2000s JWC, TVID, and CWS investigated building a 96-inch-diameter raw water pipeline to convey stored reservoir water from Hagg Lake to the JWC WTP. In addition to the 2004 Report, David Evans & Associates, in 2010, prepared the *Wetland Delineation Report*, in which they conducted an environmental assessment of the alignment and developed draft environmental-permitting documentation.

The 2004 Report identified the benefits of building a new raw water pipeline, which are documented in Table 6.5 alongside their applicability to JWC today. Many of these were predicated on the assumption that Scoggins Dam will be raised by 40 feet, thus expanding the capacity of Hagg Lake, to support future water demands. Since then, USBOR has decided that Scoggins Dam will not be raised, thus invalidating certain anticipated benefits.

Ultimately, the raw water pipeline project was abandoned due to high property-acquisition and construction costs.

Table 6.5 Benefits of a Raw Water Pipeline from 2004 Raw Water Pipeline Report

2004 Benefit	Description	Applicability to JWC Today
Raw water capacity expansion	Raising Scoggins Dam will provide additional storage water rights that can be supplied by a raw water pipeline.	No longer applicable because Scoggins Dam will not be raised to increase stored water capacity.
Water conservation and operational efficiencies and flexibilities	Raw water pipeline will reduce the routine loss of "flow-by" water that results from current "call-for-water" lag-time circumstances.	Applicable.
Tualatin river stream channel stress reduction and summer bank-overflow protection	Raw water pipeline will reduce summertime stream-bank stresses on the natural river channel between Scoggins Creek and Gales Creek.	Not as applicable since Scoggins Dam will not be raised.
Source protection, vulnerability reduction, and system security	Supply from a raw water pipeline is less vulnerable to contamination.	Applicable.
Energy Conservation and Cost Savings.	Raw water pipeline can supply the JWC WTP by gravity flow.	Applicable.
Energy efficiency and potential for small scale-turbine generation	Raw water pipeline can supply the JWC WTP by gravity flow.	Little turbine generation potential.
Water treatment and chemical cost reductions	Supply from a raw water pipeline may be less turbid than flows from the Tualatin River, requiring less treatment.	Applicable.

2004 Benefit	Description	Applicability to JWC Today
Water quality enhancement	Supply from a raw water pipeline may be less turbid and less susceptible to contamination than flows from the Tualatin River, requiring less treatment.	Applicable.
Potential TVID operational cost reductions	Potential to reduce TVID operational costs.	Applicable to potential JWC partner.
Stream flow water quality enhancements	Helps CWS meet their stream flow water quality targets, particularly temperature.	Applicable to potential JWC partner.

6.4.2 Available Supply Based on Existing Water Rights

As noted earlier, JWC currently has the following water rights:

- **Stored water rights**:
  - Maximum volume of water that can be released from a reservoir over the course of a year. Some of these water rights also have a maximum authorized flow rate associated with them.
- **Natural flow water rights**:
  - Maximum flow that can be withdrawn from a river.

The existing SHPP's location makes it capable of diverting all of JWC's stored water and natural flow water rights from the Tualatin River. However, most of JWC's natural flow water rights cannot be diverted at Scoggins Dam's municipal and industrial outlet because they are water rights for water bodies or points of diversion (POD) downstream of the dam. Thus, JWC can only divert water directly from Scoggins Dam's POD using water rights that currently have authorized PODs for or upstream of Scoggins Dam. Table 6.6 presents the water rights that can be available to JWC for withdrawal from a Scoggins Dam's POD.

The estimated annual volume available under Sain Creek's water rights shown is founded on the creek's flow data from 2001 through 2020. However, the full authorized rate is typically unavailable due to lower instream flows observed at this creek.

Meanwhile, the estimated volume available from Hagg Lake is 4,400 MG and was assumed to be equal to the contracted maximum of 13,500 acre-feet (AF) for all JWC partners. Between 2001 and 2020, the only year the actual volume available was lower than this amount was 2001 at 10,291 AF.

Note that the water available from Hagg Lake is volume limited. Regardless of the authorized rate, only 13,500 AF per year can be used under these water rights, meaning the amount of water available for diversion at Scoggins Dam during the winter months is only 5 cfs (i.e., 3 mgd) unless JWC draws on their stored water accounts during the winter. If storage is used continuously throughout the year, the mean daily volume of water available under all the rights shown in Table 6.6 will be approximately 15 mgd.

Table 6.6 Water Rights at Potential Future Scoggins Dam POD

Certificate	Application	Permit	Source	Source Type	Authorized Rate (cfs)	Estimated Annual Volume Available (AF)	Estimated Annual Volume Available (MG)
81026	S-2016	S-1136	Sain Creek	Natural flow	3	3,130	1,020
81027	S-4250	S-2443	Sain Creek	Natural flow	2		
87304	S-38447	S-35792	Scoggins Reservoir/ Henry Hagg Lake	Stored water secondary	70	13,500	4,400
93873	S-38447	S-35792	Scoggins Reservoir/ Henry Hagg Lake	Stored water secondary	N/A		
Total						**16,630**	**5,420**

Notes:
Abbreviations: MG – million gallons; N/A – not applicable.

However, a raw water pipeline constant flow rate of 15 mgd throughout the year is unrealistic, given that JWC relies on their stored water rights to meet peak summer demands and does not want to deplete their authorized stored water volume during the winter. A likelier option is for JWC to use the raw water pipeline's flow in excess of the 3-mgd natural flow water rights only in the summer or during an emergency.

JWC can use the raw water pipeline for an emergency flow of 28 mgd, which corresponds to their LOS supply goal for a Cascadia Subduction Zone (CSZ) earthquake event, for a total of 157 days or approximately five months before depleting their storage volume in Hagg Lake. An emergency flow of 42 mgd, which corresponds to JWC's LOS supply goal for a 72-year earthquake event, will last 105 days or approximately 3.5 months. The maximum withdrawal rate from Hagg Lake is 70 cfs or 45 mgd. Therefore, the maximum allowable JWC flow through the raw water pipeline will be 48 mgd, which is the sum of the 3-mgd Sain Creek natural flow water right and the maximum allowable withdrawal rate from Hagg Lake.

6.4.3 Cost Estimate

6.4.3.1 Diameter

To calculate the required diameter for the raw water pipeline and develop its cost estimate, a target capacity of 28 mgd was selected. As mentioned in Section 7.4.2, this capacity aligns with JWC's LOS goal for a CSZ event. The following assumptions were made for the hydraulic analysis used to size the pipeline:

- **Elevation in Scoggins Reservoir**:
  - High: 305 feet.
  - Low: 258 feet.
- **Elevation in the rapid mix basin at the JWC WTP**:
  - High: 197 feet.
  - Low: 195 feet.
- **Pipeline C-factor**: 145.
- **Pipeline length**: Approximately 6.5 miles or 34,300 feet.

A 40- to 42-inch pipe is the smallest-diameter pipe that can provide at least 28 mgd when Hagg Lake is at its lowest water elevation and the JWC WTP's rapid-mix water elevation is at its maximum. A 40-inch pipe can provide flows in the range of 33 to 45 mgd, depending on water elevation conditions.

6.4.3.2 Road and River Crossings

A total of seven major road, creek, and river crossings have been identified in the project area, each of which is listed in the construction cost estimate (CCE) with the assumed construction method to be used for the crossing.

The most considerable and expensive crossing will occur where the pipeline crosses the Tualatin River and wetlands area. This crossing will likely be achieved by a horizontal directional drill (HDD) of an approximately 1,000-foot length to provide both a seismically resilient design and a construction method that minimizes environmental disturbance to the river and nearby ecosystems. HDD technology is often implemented on smaller pipelines of 18 to 36 inches in size and, though possible, attempting to utilize HDD to install a 40/42-inch pipeline is on the "large-end" of the scale for this technology's applicability.

6.4.3.3 Pipe Material and Wall Thickness

The new pipeline will be of welded joint-steel construction designed to meet JWC's seismic resiliency criteria. The pipeline will be buried with a minimum cover of three feet and a typical total trench depth of approximately seven feet.

For the design of a 40-inch steel pipeline, minimum handling and deflection criteria will normally dictate a steel pipe wall of approximately 0.20 inches. Meanwhile, a thicker wall section is recommended to allow for greater strength and design resiliency at the welded joints. An overall pipe wall thickness of 5/16 inches (i.e., 0.313 inches) was assumed in the conceptual-level CCE. For this application, the steel pipeline is assumed to be mortar-lined and -coated, per the American Water Works Association's C205 requirements.

The west end of the pipeline can be constructed through areas that are perhaps less prone to liquefaction and designed using bell and spigot joints with an exterior fillet-weld system. The interior of the joint can also be fillet-welded to provide added seismic resiliency.

The eastern half of the pipeline will be constructed through an area that may be more prone to liquefaction and, thus, is assumed to be designed for a butt-welded joint system using a single outside bevel.

6.4.3.4 Cost

Table 6.7 breaks down estimated costs for this new raw water pipeline from Hagg Lake and Scoggins Dam to the JWC WTP. The conceptual-level CCE for a 40-inch-outer-diameter welded steel pipeline is approximately $60 million and includes up to seven significant road, creek, and river crossings along the alignment.

Table 6.7 Raw Water Pipeline Concept Level Construction Cost Estimate

Work Item	Cost ($ Million)
Surveying, Pipeline Testing, and Startup	$0.5
Pipeline Trench Work, Civil Site Work	$3.0
Steel Pipeline	$30.2
Major Road and River Crossings	$4.8
Estimating Contingencies	$10.0
Contractor's Overhead and Profit	$10.9
Total Estimated Probable Construction Cost	$59.5

Note that this conceptual-level CCE does not include the following project cost items:

- Local permitting, land-acquisition, and easement-acquisition costs.
- Engineering final design services.
- Any permitting fees or possible coordination services with USACE or state or local agencies relative to in-water work construction.
- Geotechnical borings and field investigations prior to design.
- Engineering services during construction, including engineering field and resident inspection services during construction.
- Unforeseen ground conditions including costs of dealing with any contaminated soils encountered.
- Owner's administration costs.

- Any costs associated with possible requirements of on-site cultural surveys and data collection in the project area.
- Any applicable local or federal taxes.

6.4.4 Permitting Considerations

The raw water pipeline's corridor crosses through the Tualatin Basin's lowlands and crosses several wetlands and streams, including the Tualatin River. Impacts on wetlands and waterways will be unavoidable, and, thus, permits will be required from USACE, DSL, DEQ, NMFS, and other federal and state agencies. In addition, land-use and floodplain authorizations that demonstrate the purpose and need of the pipeline will be required from Washington County.

6.4.5 Project Benefits

This section discusses the benefits that JWC may earn in constructing a new raw water pipeline, including water quality enhancements, operational efficiencies, energy conservation, and supply resilience.

6.4.5.1 Water Quality Enhancement

Supply through a raw water pipeline directly from Hagg Lake will decrease JWC's vulnerability to water quality events, such as high turbidity, a chemical spill, or harmful algal blooms that contaminate the Tualatin River. If a water quality event occurs, JWC can maximize flow through the raw water pipeline and minimize or eliminate flow from an intake on the Tualatin River.

Furthermore, higher water quality can lead to reduced chemical use and costs at the JWC WTP.

6.4.5.2 Operational Efficiencies

Releasing stored Hagg Lake water through the raw water pipeline will simplify JWC's operations. Currently, JWC must call for water to be released from Hagg Lake and then wait until the water reaches the SHPP before pumping it into the WTP. With a raw water pipeline, stored Hagg Lake water will flow directly to the WTP's rapid-mix basin.

6.4.5.3 Energy Conservation

A raw water pipeline will allow Hagg Lake's water to flow to the WTP by gravity, reducing the need for intake pumping and saving energy and operational costs.

6.4.5.4 Supply Resilience

The raw water pipeline can serve as a supplementary water supply to the SHPP or a new Tualatin River intake. It can be designed to be seismically resilient and sized to meet JWC's LOS goals.

6.4.6 Project Challenges

Three challenges facing a JWC raw water pipeline project are: 1) the limited availability of water rights at Scoggins Dam's POD, 2) the high cost, and 3) the need to demonstrate the pipeline's need to permitting regulatory agencies when a new intake on Tualatin River alone is sufficient to meeting JWC's goals and requirements.

Given these considerations, building a new, seismically resilient intake on the Tualatin River that can divert all of JWC's water rights and meet their LOS goals is a higher priority than building a raw water pipeline from Scoggins Dam.

6.4.7 Raw Water Pipeline Recommendations

Constructing a raw water pipeline would be very expensive and has limited benefit at this time, however, JWC wants to keep the option of a future raw water pipeline available to allow for flexibility in the future in case conditions change. As such, JWC intends to continue acquiring easements along the pipeline route. Changes that would trigger reconsideration of the raw water pipeline include 1) TVID or CWS initiating a raw water pipeline project that JWC could join, 2) stored water releases during the summer becomes a challenge due to stream-bank overtopping, or 3) increased storage capacity in Hagg Lake or natural flow water rights upstream of Hagg Lake become available to JWC.

6.5 Final Recommendations

This alternatives analysis recommends that JWC construct their own intake facility on the Tualatin River, employ the SHPP to the end of its useful life as a redundant water supply source, and not pursue the construction of a new raw water pipeline at this time.

Of the three alternatives evaluated, the overall best long-term intake solution is the construction of an intake facility on Property 2. JWC is recommended to pursue a lot line adjustment with the owners of Property 2 to append approximately six acres to the JWC WTP property.

If acquiring the proposed site on Property 2 is infeasible or if constructing a new intake is cost-prohibitive, JWC is recommended to further evaluate options to upgrade their capacity and seismic resilience through improvements to the SHPP.

Chapter 7

WTP EVALUATION

7.1 Introduction

The Joint Water Commission's (JWC) Water Treatment Plant (WTP) is a conventional WTP that withdraws water from the Tualatin River through a separate intake structure at the Spring Hill Pumping Plant. Overview information about the existing WTP components and their general condition is presented in Chapter 2. This chapter focuses on an alternatives analysis for disinfection improvements and incorporates recommendations from this evaluation with previously identified WTP projects to provide a comprehensive list of improvements for the JWC WTP. Preferred alternatives for disinfection improvements are incorporated into the improvements, with the remaining improvements predominantly taken from recommended projects in JWC's 2018 *Water Treatment Facility Plan* (2018 Facility Plan). Additional WTP-specific projects were identified from this planning effort after discussions with JWC WTP staff (identified in Chapter 2), the water quality blending evaluation (provided in Appendix G), and the WTP intake evaluation (Chapter 6). The final WTP improvement plan provides a list of projects for consideration in the development of the capital improvement program presented in Chapter 9 of this master plan.

7.2 Disinfection Improvements Alternatives Analysis

7.2.1 Purpose and Needs Statement

JWC currently utilizes chlorine gas for disinfection and other treatment process purposes at the WTP. Chlorine gas is a highly toxic chemical that poses safety challenges and has ongoing operations and maintenance (O&M) challenges associated with the chlorine scrubber system and other safety regulations. There are limited chlorine suppliers available in the region, and one major supplier was unable to supply chlorine to utilities throughout the Pacific Northwest in the summer of 2021. The lack of viable sources for chlorine gas poses resilience and reliability challenges for the system and creates a non-competitive cost environment. Additionally, the existing chlorine gas treatment system lacks seismic resilience.

The purpose of this disinfection alternatives analysis is to identify and evaluate improvements that replace the existing chlorine gas system with a new disinfection system. The new system must provide robust and reliable treatment to meet existing treatment process needs and be seismically resilient to meet JWC’s level of service goals.

7.2.2 Alternatives Identification

Five alternatives were identified for consideration:

- **Chlorine Gas:** Seismically retrofitting the existing chlorine gas system or constructing a new chlorine gas system in a new seismically resilient building.

- **Bulk Sodium Hypochlorite (Bulk Hypochlorite):** Construct chemical storage and metering facilities to use liquid bulk sodium hypochlorite (12.5 percent solution strength) for disinfection, with all sodium hypochlorite delivered to the WTP.
- **Full On-Site Hypochlorite Generation (OSHG) with Full Bulk Sodium Hypochlorite Backup (Full OSHG):** Construct an OSHG system to generate dilute liquid sodium hypochlorite (0.8 percent solution strength) on-site from delivered salt. The OSHG system is designed to meet disinfection requirements across all plant flow ranges. On-site chemical storage can also be used to accept deliveries of bulk sodium hypochlorite as a fully redundant backup disinfectant source.
- **Bulk Sodium Hypochlorite with Backup OSHG (Bulk Hypochlorite with Backup OSHG):** Construct chemical storage and metering facilities sized to use bulk liquid sodium hypochlorite to meet all disinfection requirements across all plant flow ranges. This alternative also includes a smaller, backup OSHG system sized only to meet wintertime average day demands (ADD) at average chemical doses.
- **Bulk Sodium Hypochlorite with Provisions for Future Conversion to Full On-Site Hypochlorite Generation (Bulk Hypochlorite Future OSHG):** Construct chemical storage and metering facilities designed to use bulk liquid sodium hypochlorite to meet all disinfection requirements across all plant flow ranges. Design the chemical facilities with adequate space and provisions to accommodate installation of an OSHG system in the future.

7.2.3 Existing Chlorine Gas System

JWC's existing chlorine gas system is housed in the main operations building of the WTP. Nine one-ton cylinders of chlorine gas are stored on-site, with six cylinders active, feeding chlorine gas to seven chlorinators. The chlorine cylinders and chlorinators are in adjacent rooms within the main operations building at the WTP. A chlorine scrubber is housed outside the operations building in a separate enclosure. At a cost of $1,500 per ton (as of May 1, 2022), chlorine gas supply for the current system costs approximately $161,000 per year, assuming an ADD of 32 million gallons per day (mgd) and average chlorine dose of 2.2 milligrams per liter (mg/L) (see Section 7.2.5 for discussion of current demands and chlorine doses).

The 2008 *Water Treatment Plant Seismic Evaluation* identified significant seismic risk for the operations building with moderate to severe earthquake damage anticipated for all seismic events evaluated. The 2015 *Capital Improvement Program Update* determined that seismic mitigations to the existing operations building were impractical.

Should JWC continue to utilize the existing chlorine gas system moving forward with no mitigations, it is anticipated a seismic event would require complete replacement of the chlorine gas system including a new facility to house the system due to extensive damage to the operations building. Under this no action scenario, installation of a new chlorine gas facility would be approximately $5.5 million.

7.2.4 Alternatives Screening

The five identified alternatives were screened for viability, safety, high-level cost estimate, and reliability. Table 7.1 summarizes the preliminary screening. Note, no separate entry is shown for the Bulk Hypochlorite Future OSHG alternative as it is equivalent to the Bulk Hypochlorite alternative in the screened categories in the near-term until OSHG is installed and equivalent to Full OSHG in the future.

Table 7.1 Disinfection Alternatives Screening

Category	Chlorine Gas	Bulk Hypochlorite[2]	Full OSHG	Bulk Hypochlorite with Backup OSHG
Viability	• Currently used at the JWC WTP.	• Widely used in the industry. • Simple equipment.	• Established technology with increasing frequency of use in the industry. • More complex equipment that could be affected by long lead times and outages for replacement parts.	• Bulk hypochlorite and OSHG systems are both viable disinfectant sources. • Using OSHG as a backup supply is not common. Different chemical strengths and system idle time could complicate operations.
Safety	• Highest safety risk. • Toxic gas poses risks to WTP staff and surrounding communities (Forest Grove, Cornelius, and Dilley).	• Lower safety risk. • Bulk hypochlorite is a hazardous, corrosive liquid.	• Lowest safety risk. • Dilute sodium hypochlorite is not classified as a hazardous chemical.	• Lower safety risk. • Bulk hypochlorite is a hazardous, corrosive liquid.
High-Level Capital Cost	• Lowest Cost	• Lower Cost	• Highest Cost	• High Cost
High-Level O&M Cost	• Lowest Cost	• High Cost	• Lower Cost	• High Cost[1]
Reliability	• Least reliable. • Vulnerable to supply disruptions with limited suppliers. • Concerns regarding ability to reliably transport chlorine gas post-seismic event.	• Limited reliability. • Limited regional producers leave system vulnerable to chemical supply disruptions, particularly post-seismic event. • Supply can be transported from outside the region if regional supply disruptions occur.	• Most reliable. • Salt can be transported from long distances if local supply disruptions occur. • Additional redundancy provided through ability to accept delivered bulk sodium hypochlorite as a backup. • Seismic resilience of generation equipment unknown; vendors unable to confirm.	• More reliable. • Backup OSHG mitigates concerns with chemical supply availability. • Reliant on outside bulk hypochlorite supply reliability and redundancy to meet peak demands.

Notes:
(1) O&M costs dependent on how often OSHG is used in place of bulk hypochlorite. Highest cost O&M alternative if OSHG is only rarely used.
(2) Bulk Hypochlorite Future OSHG is equivalent to Bulk Hypochlorite information shown in the near-term until OSHG is installed and equivalent to Full OSHG in the future.

Based on this preliminary screening, chlorine gas was removed from further consideration due to the significant safety concerns and limited resilience and reliability of the chlorine gas supply. Additionally, the lack of suppliers and competition makes contractual agreements vulnerable to price increases outside contractual limits. Four alternatives were carried forward for detailed evaluation:

- Bulk Hypochlorite.
- Full OSHG.
- Bulk Hypochlorite with Backup OSHG.
- Bulk Hypochlorite Future OSHG.

7.2.5 Common Design Criteria for Alternatives Sizing

7.2.5.1 Water Treatment Plant Production

Table 7.2 summarizes the current and projected flow rates for the JWC WTP that were considered in this analysis.

Table 7.2 Design Flow Rates

Parameter	Current Operations (2016 - 2021)(1) (mgd)	2042 Demands (Initial System Sizing)	Build-Out
Minimum Demand	15 mgd	15 mgd	30 mgd
Winter Day Demand	25 mgd(2)	33 mgd(3)	50 mgd
Average Day Demand	32 mgd	42 mgd(3)	64 mgd(5)
Maximum Day Demand	63 mgd	83 mgd(4)	125 mgd
Peak Production	85 mgd	85 mgd	125 mgd

Notes:

(1) From 2016 - 2021 finished water production data.

(2) Average winter day demand found to be 77 percent of average day demand from analysis of 2016 - 2021 finished water production data. Ratio of winter day demand to average day demand assumed to remain consistent in the future.

(3) Average day demand and average winter day demand increased by 32 percent from current value to estimate 2042 demand. Percentage increase matches 32 percent increase in maximum day demand between current 63 mgd and 2042 projection of 83 mgd presented in Chapter 5.

(4) Demand projection presented in Chapter 5 of this plan.

(5) Estimated using a maximum day demand to average day demand peaking factor of 1.96 calculated from current operations. Ratio assumed to remain consistent in the future.

Current operations and existing demands were characterized using operating data from 2016 to 2021. Future demands for 2042 and build-out were projected for equipment sizing for each disinfection alternative. Initial system sizing was based on 2042 demands to be consistent with the 20-year planning period of this master plan. Build-out demands were considered to understand the size and space needed to meet ultimate plant capacities. Ultimate build-out capacity was taken to be 125 mgd.

7.2.5.2 Applied Chlorine Doses

Historical plant data from 2016 to 2021 was reviewed to identify the typical range of applied chlorine doses across the entire treatment process:

- Minimum: 1.2 mg/L.
- Average: 2.2 mg/L.
- Maximum 3.4 mg/L.

This range of chlorine doses was used in conjunction with the plant flow rates presented in Table 7.2 to size key components for each of the alternatives. No changes to plant operations and chlorine dosing strategies in the future were considered.

7.2.5.3 Chemical Storage

Bulk Hypochlorite

Required chemical storage volumes were developed starting with typical chemical storage design criteria in the industry, with the largest storage volume governing tank size selection:

- Average flow and average dose: 30 days (23,100 gallons at 42 mgd and 2.2 mg/L).
- Average flow and maximum dose: 14 days (16,700 gallons at 42 mgd and 3.4 mg/L).
- Maximum flow and average dose: 14 days (21,800 gallons 85 mgd and 2.2 mg/L).

Chemical storage design criteria generally assume all tanks are 100 percent full when calculating the total available days of storage. For sizing of chemical storage in this analysis, calculations assumed tanks were only 80 percent full to account for variabilities in operations and chemical deliveries that result in storage tanks rarely being 100 percent full.

Full OSHG

Chemical storage criteria for OSHG differ because the sodium hypochlorite is produced on-site and is produced at low strength. Maintaining weeks to a month of storage would require substantial storage volumes. For OSHG, hypochlorite storage volume was sized to ensure a minimum of one day of storage at peak flow and peak chemical dose (35,500 gallons at 85 mgd and 3.4 mg/L). Generators were sized to provide a redundant generator at peak day flows and average chemical dose to accommodate maintenance or unexpected failures.

Salt storage was sized assuming the same storage criteria for the bulk hypochlorite (i.e., 30 days of storage at average flow and average dose), including the 80 percent full condition, with the largest storage volume governing (35 tons at 42 mgd and 2.2 mg/L).

Bulk Hypochlorite with Backup OSHG

Storage requirements for operating with delivered bulk sodium hypochlorite were kept the same as the bulk hypochlorite alternative.

For the backup OSHG system, salt storage was sized to provide 30 days of storage at average winter day demand and average dose (27 tons at 33 mgd and 2.2 mg/L chlorine dose). With the OSHG system serving as a backup, the 80 percent full condition was not applied.

Bulk Hypochlorite Future OSHG

Chemical storage requirements were kept the same as the bulk hypochlorite alternative for initial operation using bulk hypochlorite. Storage requirements for the future OSHG were assumed to be the same as the full OSHG system to ensure sufficient space is available for future OSHG operation.

7.2.5.4 Chemical Metering

Chlorine is currently dosed to seven separate locations at the JWC WTP. For conceptual space planning purposes, chemical metering pumps were assumed for all alternatives. Chemical metering should be further refined during detailed design, which may opt for a pressurized loop system as an alternative to installing and maintaining this large number of metering pumps. Table 7.3 summarizes all dosing locations and the corresponding number of chemical metering pumps assumed.

Table 7.3 Assumed Chemical Metering

Dosing Location / Function	Number of Pumps	Notes
Rapid Mix	2 (1 duty, 1 standby)	
Settled Water	3 (2 duty, 1 standby)	Dedicated pump for each current dosing point, one to filters 1 - 14 and one to filters 15 - 16. Single redundant pump for either dosing point.
Combined Filter Effluent	4 (2 duty, 2 standby)	Dedicated duty and standby pumps for current dosing points to filters 1 - 14 and filters 15 - 16.
Finished Water	3 (2 duty, 1 standby)	Dedicated pump for each finished water pump station. Single redundant pump for either dosing point.
Future	3	Pumps for potential additional future dosing points.
Total Number of Chemical Metering Pumps	12 (Current) 15 (Future)	One additional tank transfer pump is assumed for transferring chemicals between storage tanks. Tank transfer pump not shown in figures in this chapter.

A total of 12 metering pumps were assumed to meet initial chemical metering needs with space reserved for three additional pumps in the future. This includes redundant metering pumps provided for each dosing location, with additional redundancy provided for the combined filter effluent dosing location, as this is the most critical chemical dosing point for regulatory disinfection compliance.

7.2.6 Disinfection Alternative Details and Conceptual Layouts

The following sections provide an overview of each disinfection alternative evaluated, including design criteria and conceptual building layouts.

7.2.6.1 Bulk Hypochlorite

Bulk sodium hypochlorite is a commercially available solution commonly supplied in concentrations ranging from 10 to 15 percent. For these systems, bulk sodium hypochlorite is delivered to the WTP site where it is stored in indoor tanks (typically polyethylene or fiber-reinforced plastic). It is subsequently metered to injection points throughout the process, typically with metering pumps. Equipment requirements for bulk hypochlorite systems are relatively minimal, mainly the chemical storage tanks and the metering pumps.

While bulk hypochlorite systems are relatively simple, there are a few key considerations for chemical storage and operation. Bulk hypochlorite is classified as a hazardous chemical and requires secondary containment for the chemical storage tanks. Additionally, bulk hypochlorite strength degrades over time in storage. The rate and degree of degradation is affected by several factors including solution strength, heat, light, pH, and storage time. Degradation of sodium hypochlorite also produces chlorate, which may be regulated in the future as revisions to the Microbial and Disinfection Byproducts Rules are considered (see Chapter 4). While degradation of bulk hypochlorite cannot be avoided, it can be managed by controlling storage conditions, securing high-quality chemicals, and/or dilution.

For sizing bulk hypochlorite storage, storage strength was assumed to be 10 percent to reflect degradation from the typical delivered 12.5 percent strength. The degradation in solution strength must also be carefully monitored to ensure adequate and effective dosing is maintained throughout the treatment process.

Preliminary design criteria for the bulk hypochlorite alternative at JWC are shown in Table 7.4.

Table 7.4 Bulk Hypochlorite Design Criteria

Parameter	Unit	Initial Design	Future Build-out
Number of Hypochlorite Storage Tanks	No.	3	4
Hypochlorite Storage Tank Volume (ea)$^{(2)}$	Gallons	10,000	12,000
Total Hypochlorite Storage Volume	Gallons	30,000	48,000
Days of Bulk Hypochlorite Storage at Average Flow and Average Dose$^{(1)}$	Days	31	33
Metering Pumps	No.	12	15
Transfer Pumps	No.	1	1

Notes:
Abbreviations: ea - each; No. - number.
(1) Assumes tanks are 80 percent full.
(2) Nominal tank storage volumes shown.

Three 10,000-gallon tanks are sufficient to meet chemical storage requirements for 2042 initial design flow rates, but it is anticipated that larger tanks will be needed in the future to meet chemical storage goals. Given that ultimate build-out is outside the 20-year planning horizon and the lifespan of chemical storage tanks is on the order of 10 to 20 years, it is assumed JWC would transition to larger tanks as demands increase in the future. Chemical storage pads would need to be sized to accommodate both tank sizes and overall building design would need to accommodate the difference in height between the two tank sizes.

Figure 7.1 shows a conceptual building layout to house all equipment needed for a bulk hypochlorite system. The building footprint is sized with space for all future build-out equipment. Future equipment is shown with dashed lines. Note, tank dimensions shown in Figure 7.1 are based on actual dimensions for commercially available tanks with the nominal storage capacities identified in Table 7.4.

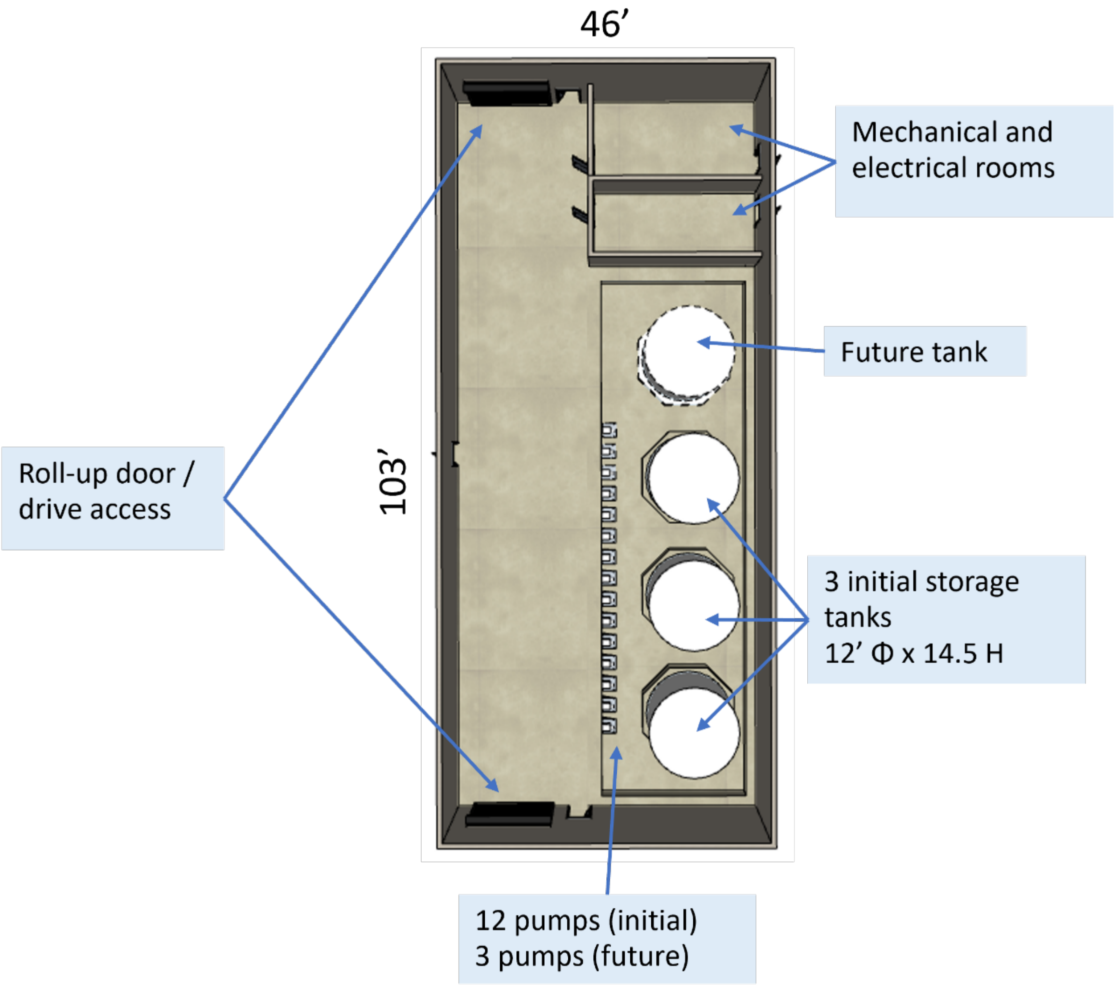


Figure 7.1 Bulk Hypochlorite Conceptual Building Layout

Total building footprint is approximately 4,700 square feet, which is one-third of the total footprint allocated in the 2018 Facility Plan for the future chemical building. Building footprint includes space for separate mechanical and electrical rooms with dedicated egress from the building, chemical tanks and metering pumps, and drive access through roll-up doors to facilitate tank removal.

7.2.6.2 Full On-Site Hypochlorite Generation

OSHG systems utilize water, salt, and electricity to generate a weak (0.8 percent) sodium hypochlorite solution. Salt is dissolved in water and stored as a saturated salt brine solution in brine tanks. The salt solution from the brine tanks is further diluted with a softened water stream before flowing through the generators. Electricity is applied to electrolytic cells in the generators to convert the dilute sodium chloride solution into a weak sodium hypochlorite solution. During this conversion hydrogen gas is also formed as a byproduct, but it is diluted to safe levels and vented to the atmosphere by blowing air through the system. Generated hypochlorite solution is stored in tanks and metered to dosing points.

Chemical metering and storage for OSHG is comparable to bulk hypochlorite with a few exceptions. First, metering pump capacities must be much larger due to the difference in solution strength. Second, dilute hypochlorite is a stable solution that does not degrade like bulk hypochlorite, reducing the need to monitor degradation and solution strength. The lack of degradation also results in less chlorate formation compared to bulk hypochlorite, though bromate can be formed from bromide present in the salt. Bromate formation can primarily be limited by securing high-quality, low-bromide salt.

OSHG systems are significantly more complex than bulk hypochlorite systems and commonly include the following major equipment, many of which are shown in the example OSHG process flow diagram in Figure 7.2:

- Generator units with electrolytic cells.
- Rectifiers to convert alternating current power to direct current (DC) power.
- Brine storage tanks.
- Hypochlorite storage tanks.
- Water softening units for feed water to the generators and brine tank.
- Heating units to maintain optimal water temperature.
- Hydrogen gas dilution blowers.

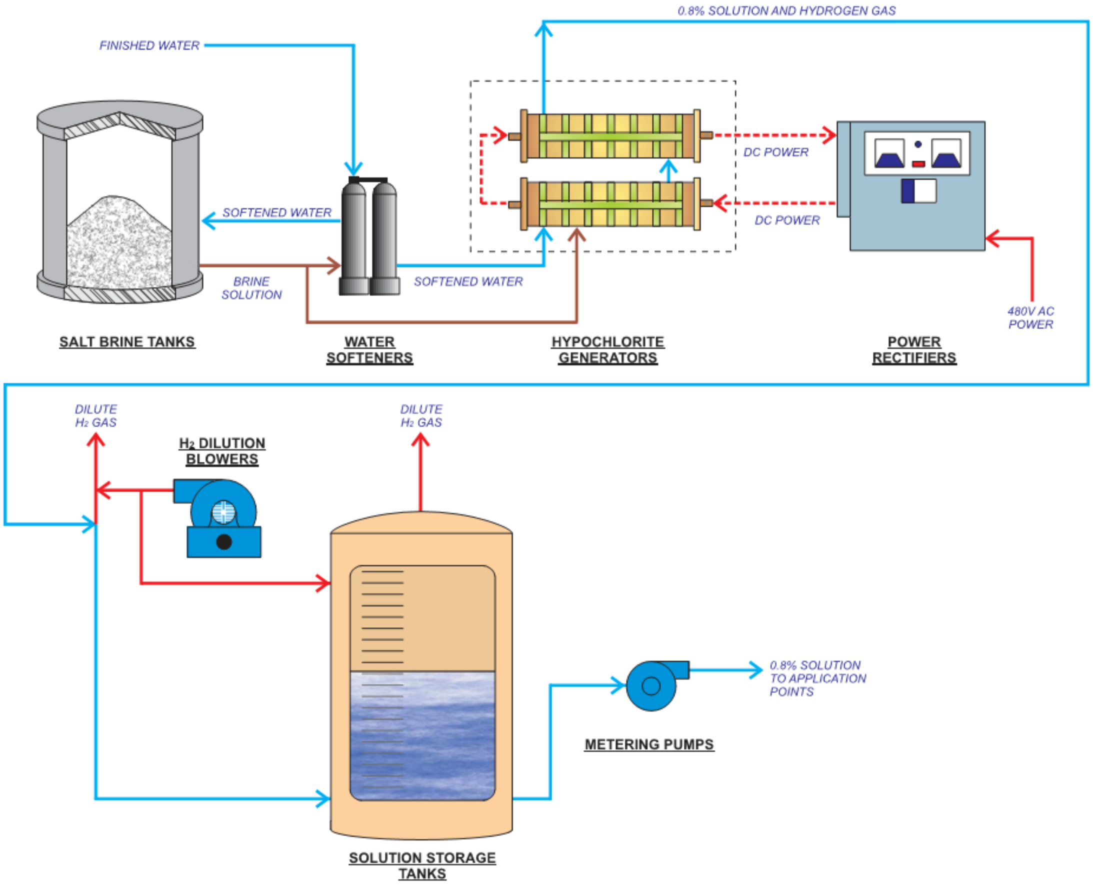


Figure 7.2 Example OSHG Process Flow Diagram

Preliminary design criteria for the full OSHG alternative at the JWC WTP are shown in Table 7.5.

Table 7.5 Full OSHG Design Criteria

Parameter	Unit	Initial Design	Future Build-out
Number of Generators	No.	3 (2 duty, 1 standby)	4 (3 duty, 1 standby)
Generator Capacity (ea)	lb/day	1,000	1,000
Total Generation Capacity (all generators online)	lb/day	3,000	4,000
Number of Hypochlorite Storage Tanks	No.	3	4
Hypochlorite Storage Tank Volume (ea)(4)	Gallons	12,000	12,000
Total Hypochlorite Storage Volume	Gallons	36,000	48,000
Days of 0.8 Percent Hypochlorite Storage at Peak Flow and Maximum Dose(3)	Days	1.0	0.9
Days of 0.8 Percent Hypochlorite Storage at Average Flow and Average Dose(1)	Days	3.2	2.8
Number of Brine Tanks	No.	2	3
Brine Tank Size (ea)	Tons	25	25
Total Salt Storage Capacity	Tons	50	75
Salt Storage at Average Flow/Average Dose(2)	Days	35	34
Metering Pumps	No.	12	15
Transfer Pumps	No.	1	1

Notes:
Abbreviations: lb/day - pounds per day.
(1) Assumes tanks are 100 percent full. Chemical storage volumes shown here represent storage under normal operations. Storage volume requirements based on peak flow and maximum dose.
(2) Assumes brine tanks are 80 percent full.
(3) Chemical storage volumes designed around this storage criterion.
(4) Nominal tank storage volumes shown.

Providing three 1,000-lb generators in a 2+1 configuration ensures sufficient capacity to meet hypochlorite production requirements at peak flows and average chemical doses with one generator offline. To meet peak flows and peak chlorine doses all generators would be needed.

Total tank storage volumes are slightly larger than for the bulk hypochlorite alternative (36,000 gallons versus 30,000 gallons for the initial design). As discussed in the chemical storage design criteria, tank volumes for full OSHG are sized to provide one day of storage at maximum flow and maximum chemical dose. During average demands and doses this provides approximately three days of storage.

If the entire OSHG system were offline, JWC would have sufficient chemical storage to accept bulk hypochlorite deliveries and operate using bulk hypochlorite. Chemical metering systems would need to be designed to account for the wide range of flow rates required for the differing solution strengths to operate in this manner. Alternatively, during an emergency, bulk hypochlorite could be delivered to one tank and diluted down to a lower strength in the remaining tanks.

Figure 7.3 shows a conceptual building layout to house all equipment needed for the full OSHG alternative. The building footprint is sized with space for all future build-out equipment. Future equipment is shown with dashed lines. Note, tank dimensions shown in Figure 7.3 are based on actual dimensions for commercially available tanks with the nominal storage capacities identified in Table 7.5.

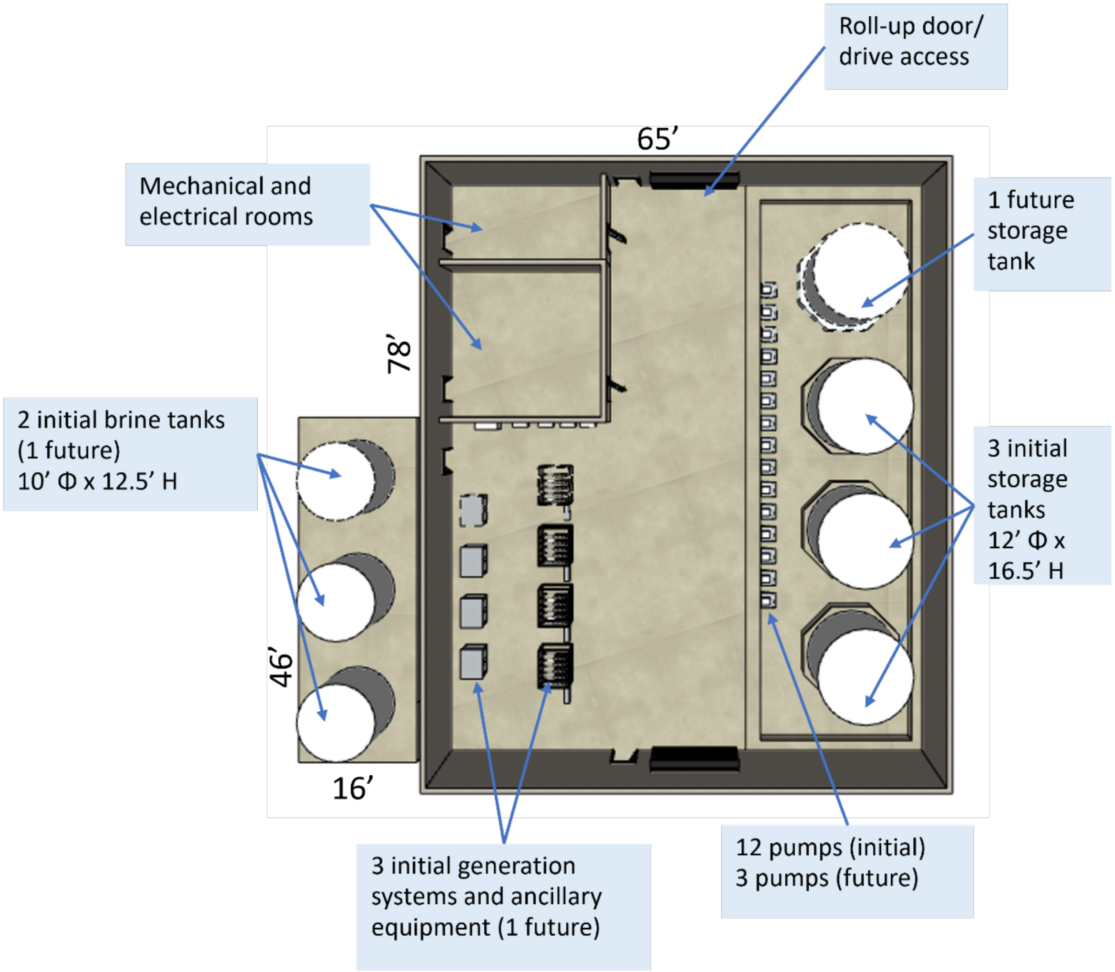


Figure 7.3 Full OSHG Conceptual Building Layout

Total building footprint for the hypochlorite tanks and OSHG equipment is approximately 5,100 square feet, larger than the bulk hypochlorite building and more than one-third of 2018 Facility Plan chemical building. The building footprint is slightly larger than the than bulk hypochlorite alternative and is reconfigured to accommodate the generation equipment. Additional space is required to house the brine tanks, which are shown in this conceptual layout on an exterior concrete pad.

The conceptual layout includes space for separate mechanical and electrical rooms with dedicated egress from the building, chemical tanks and metering pumps, and drive access through roll-up doors to facilitate tank replacement. A larger electrical room is shown to account for the increased equipment for the OSHG system.

The conceptual layout shown includes secondary chemical containment for the storage tanks even though dilute hypochlorite is not considered to be a hazardous chemical. Designing the building in this manner ensures that the secondary containment would be provided, as required, in the event JWC were to accept deliveries of bulk hypochlorite.

7.2.6.3 Bulk Hypochlorite with Backup On-Site Hypochlorite Generation

The bulk hypochlorite with backup OSHG alternative is similar to the bulk hypochlorite alternative, but with a smaller OSHG system to provide some redundancy and backup in the event of supply issues with bulk sodium hypochlorite. For this analysis, the backup OSHG system was assumed to meet only winter average day demands at average chemical doses. Bulk sodium hypochlorite would still be required to meet peak summer demands without curtailment.

Although the backup OSHG system was sized to meet winter average demands with 30 days of salt storage, the system could likely meet disinfection needs for a large portion of the winter months. Further study is needed to develop an operational strategy for this system. Leaving the OSHG system idle for large portions of the year may not be recommended by OSHG system manufacturers. Should the backup OSHG system need to be operated while also using bulk hypochlorite, this would be operationally challenging. Feeding dilute and bulk hypochlorite at the same time would result in highly variable chemical solution strength, making effective dose control difficult. A more feasible strategy would be to maintain a dedicated tank for the dilute hypochlorite and periodically switch between feeding 0.8 and 12.5 percent hypochlorite. Even so, this would require careful sizing of metering equipment and attention to dosing controls to account for the differences in solution strength when switching between the two.

Table 7.6 summarizes chemical storage design criteria for the bulk with backup OSHG alternative. Chemical storage is consistent with the bulk hypochlorite alternative as bulk sodium hypochlorite will be the primary source of disinfectant chemical. The number of chemical metering pumps would be unchanged for this alternative, matching the total of 12 pumps initially and 15 pumps at build-out shown in Table 7.4 and Table 7.5.

Table 7.6 Bulk with Backup OSHG: Bulk Hypochlorite Storage Design Criteria

Parameter	Unit	Initial Design	Future Build-out
Number of Hypochlorite Storage Tanks	No.	3	4
Hypochlorite Storage Tank Volume (ea)(2)	Gallons	10,000	10,000
Total Hypochlorite Storage Volume	Gallons	30,000	40,000
Days of Bulk Hypochlorite Storage at Average Flow and Average Dose(1)	Days	33	29

Note:
(1) Assumes tanks are 80 percent full.
(2) Nominal tank storage volumes shown.

Table 7.7 presents design criteria for the backup OSHG system. Generators are sized only to meet winter average day demands at average chlorine dose without redundancy because the OSHG serves as a backup system. At winter day average demands and average chemical doses, the storage tanks provide approximately three days of storage.

Table 7.7 Bulk with Backup OSHG: Backup On-Site Hypochlorite Generation Design Criteria

Parameter	Unit	Initial Design	Future Build-out
Number of Generators	No.	2 (2 duty, 0 standby)	3 (3 duty, 0 standby)
Generator Capacity (ea)	lb/day	400	400
Total Generation Capacity (all generators online)	lb/day	800	1,200
Days of 0.8 Percent Sodium Hypochlorite Storage at Winter ADD and Average Dose(1)	Days	2.8	2.4
Number of Brine Tanks	No.	1	2
Tank Size (ea)	Tons	35	35 / 20
Total Salt Storage Capacity	Tons	35	55
Salt Storage at Winter Average Flow and Average Dose(1)	Days	32	33

Notes:
(1) Assumes tanks are 100 percent full.

Figure 7.4 shows a conceptual layout for this alternative. Like the other conceptual layouts, the footprint is sized to accommodate all initial and future build-out equipment. Future equipment is shown with dashed lines. Note, tank dimensions shown in Figure 7.4 are based on actual dimensions for commercially available tanks with the nominal storage capacities identified in Table 7.6.

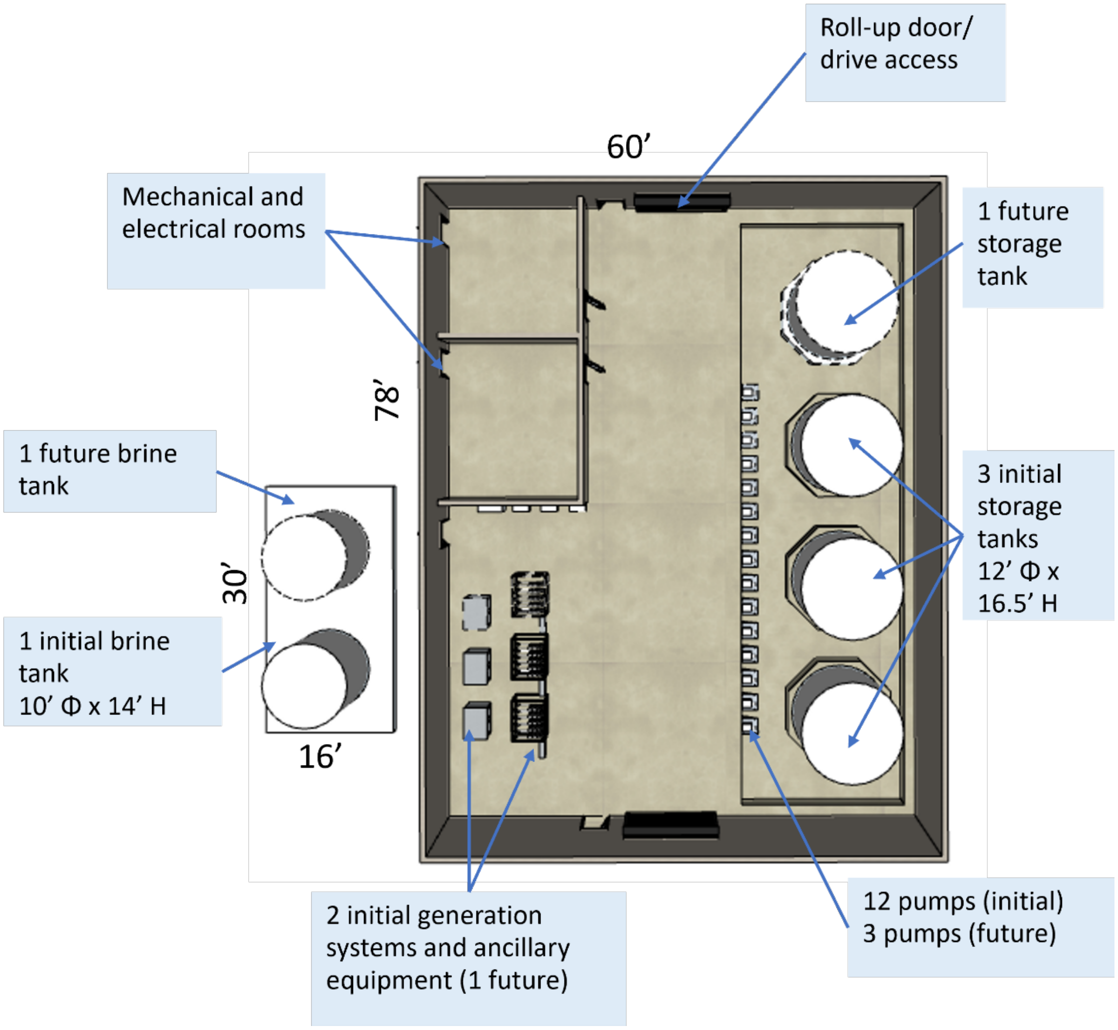


Figure 7.4 Bulk with Backup OSHG Conceptual Building Layout

This conceptual layout includes a 4,700 square-foot building footprint to house the hypochlorite storage tanks and OSHG equipment. An additional 500 square feet is reserved for the outdoor brine tank pad. Overall building footprint is only slightly reduced compared to the full OSHG alternative, as much of the footprint is needed for the storage tanks. At this conceptual stage for building layouts, reducing the size of the OSHG system and number of generators provides minimal footprint savings. Like the other layouts shown, the conceptual layout includes space for separate mechanical and electrical rooms with dedicated egress from the building, chemical tanks and metering pumps, and drive access through roll-up doors to facilitate tank removal.

7.2.6.4 Bulk Sodium Hypochlorite with Provisions for Future Conversion to Full On-Site Hypochlorite Generation

The bulk hypochlorite with provisions for future conversion to OSHG is like the bulk hypochlorite alternative for initial construction and operation. Design and sizing criteria for the bulk hypochlorite system were consistent with the bulk hypochlorite alternative (Table 7.4) as bulk hypochlorite will be used during initial operation. Design criteria and flow rates for sizing of the OSHG system will be dependent on the timing of OSHG construction. For this analysis and space planning considerations, it was assumed the OSHG system would be the same as the full OSHG alternative.

Figure 7.5 shows a conceptual layout for the bulk hypochlorite with provisions for future conversion to OSHG. Like the other conceptual layouts, the footprint is sized to accommodate all initial and future build-out equipment. Future equipment is shown with dashed lines.

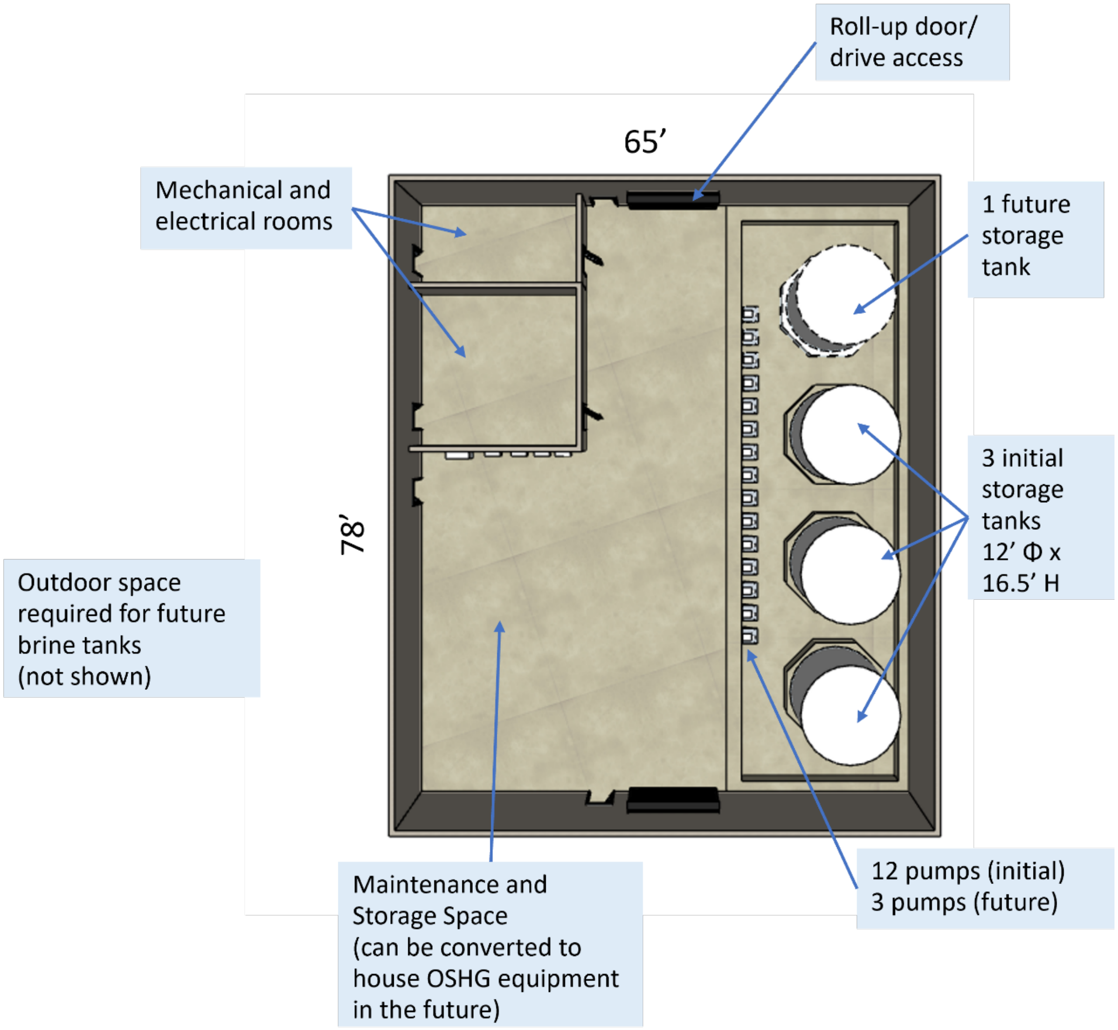


Figure 7.5 Bulk with Provisions for Future Conversion to OSHG Conceptual Building Layout

For this alternatives analysis, the conceptual layout matches the 5,100 square-foot building footprint for full OSHG (shown in Figure 7.3). This ensures sufficient space is provided to house the bulk hypochlorite storage tanks and equipment for operation in the near-term while preserving space for future OSHG equipment. In the near-term the building can provide maintenance and storage space that would be converted in the future. Like the other layouts shown, the conceptual layout includes space for separate mechanical and electrical rooms with dedicated egress from the building, chemical tanks and metering pumps, and drive access through roll-up doors to facilitate tank removal.

7.2.7 Additional Considerations

The following sections summarize additional considerations for the evaluated disinfection alternatives.

7.2.7.1 Regional Bulk Hypochlorite Supply

There are multiple distributors of bulk hypochlorite in the region, including JWC's existing chlorine gas supplier, Jones Chemical. Additional distributors include Univar, Brenntag, and Northstar. Local producers of bulk hypochlorite in the Pacific Northwest rely on chlorine gas to produce bulk hypochlorite, making these producers vulnerable to a disruption like the one that occurred in summer 2021. However, suppliers can provide bulk hypochlorite from outside the local Portland region, including Utah, California, and Wisconsin. Bulk hypochlorite provides more resilience against supply disruptions than chlorine gas because there are more distributors that will deliver bulk hypochlorite than chlorine gas. Distributors outside the Pacific Northwest have indicated that they will not deliver chlorine gas to JWC facilities.

Figure 7.6 shows the locations of domestic manufacturers and suppliers of chlorine gas and Figure 7.7 shows the locations of domestic manufacturers and suppliers of bulk hypochlorite. As shown in the figures, there are many more domestic manufacturers and suppliers of bulk hypochlorite than chlorine gas.

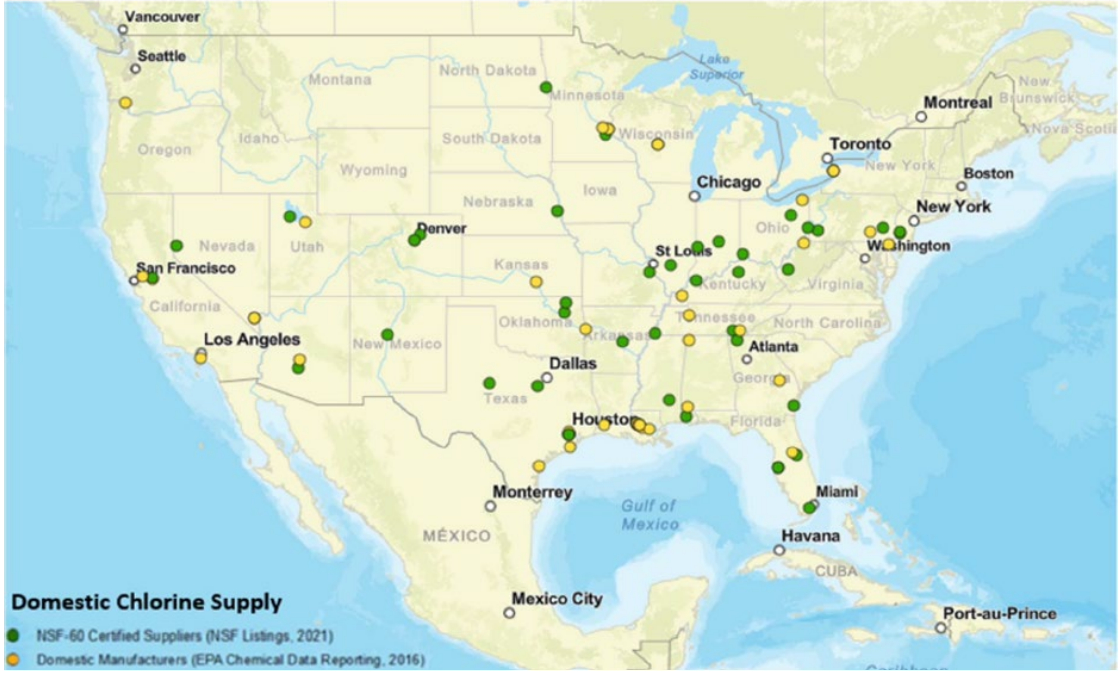


Figure 7.6 Domestic Chlorine Gas Production and Supplier Facilities (Source: US Environmental Protection Agency)

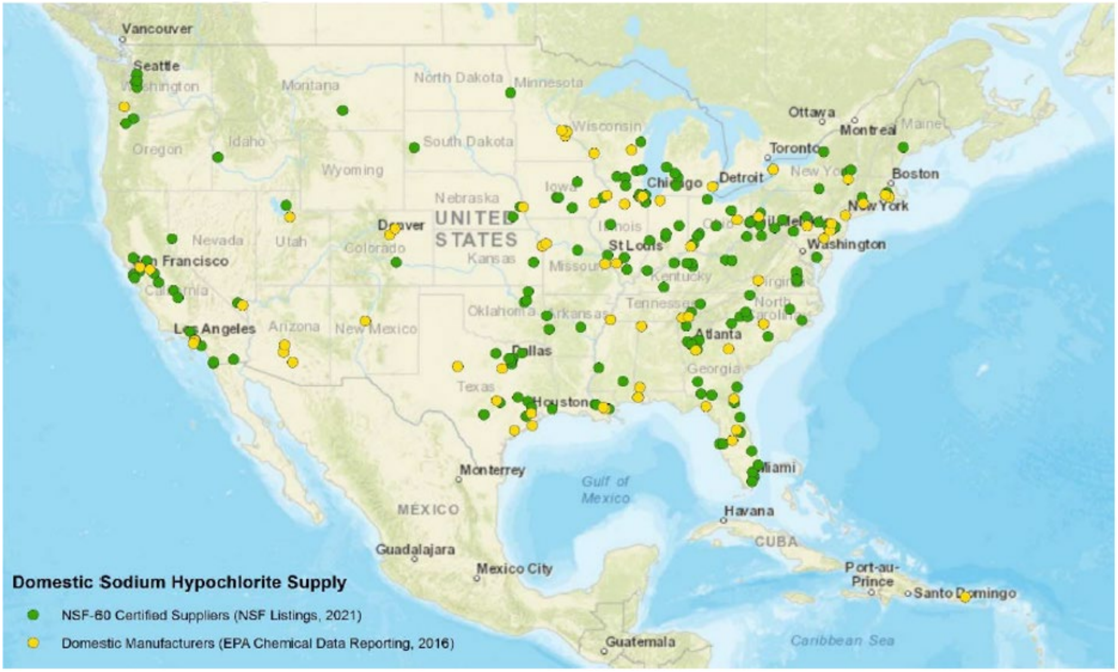


Figure 7.7 Domestic Hypochlorite Production and Supplier Facilities (Source: US Environmental Protection Agency)

If bulk hypochlorite is selected for chlorine replacement, then it is recommended that JWC consider having multiple contracts for delivery of bulk hypochlorite in the event of an emergency or supply disruptions to mitigate the following issues:

- The corrosive nature of bulk hypochlorite requires it be delivered in special tanker trucks with fiberglass reinforced tanks, which could limit the number of available trucks for chemical delivery.
- Without a contract in place, suppliers outside the region may prioritize delivery to local customers, limiting the availability for shipping long distances.

7.2.7.2 Regional Salt Supply

OSHG uses National Sanitation Foundation (NSF)-60 certified salt. There are no local suppliers in Oregon or Washington that produce the salt, but multiple major suppliers have facilities in the western United States, including Morton, Compass Minerals, and Cargill. The majority of the salt is harvested at facilities in Utah but is distributed and purified at facilities across the United States. The major suppliers have facilities in the following locations in the western United States:

- Morton: Grantsville, UT; Long Beach, CA; Glendale, AZ.
- Compass Minerals: Ogden, UT.
- Cargill: Newark, CA; Timpie, UT.

Brenntag and Cascade Columbia are local suppliers of salt for OSHG. Brenntag typically supplies Morton or Cargill salt from Utah locally for OSHG. Cascade Columbia typically supplies Cargill Diamond Crystal Salt, but can also supply salt Morton salt from Newark, CA. Step Saver, an additional distributor located in Salt Lake City, UT, can also supply salt from Morton or Compass Minerals to Oregon. Given the distance, Step Saver typically requires a minimum of one week lead time on salt deliveries.

From discussions with the distributors, there have not been documented issues meeting salt delivery needs in the past. For additional supply reliability, Brenntag offers the option to store super sacks of salt in their regional warehouse as an emergency supply source, provided the supply is rotated periodically.

Multiple suppliers and facilities exist across the western United States that can provide the salt for OSHG, providing a greater level of chemical supply reliability and redundancy compared to chlorine gas or bulk sodium hypochlorite.

7.2.7.3 Impacts of OSHG on Backup Generator Operation

One important consideration with OSHG is the electrical load associated with the generators. Additional evaluation will be needed during preliminary design to determine how to incorporate OSHG into the existing WTP electrical infrastructure. For this alternatives analysis, a high-level evaluation was conducted to determine the impact these additional loads may have on production rates when on generator power.

JWC installed two 2.5-megawatt (MW) backup generators in 2016 that were sized to ensure JWC could produce 37.5 mgd (50 percent of plant capacity at the time of construction) in the event of a utility power outage. Historical operating data while the plant ran on generator power was analyzed to understand if the additional electrical loads for OSHG would limit JWC from meeting this capacity.

To determine the impacts on production capacity while running on generator power, operating data from September 2020 was evaluated. Figure 7.8 shows finished water flow rates and corresponding electrical loads on the generators during this event.

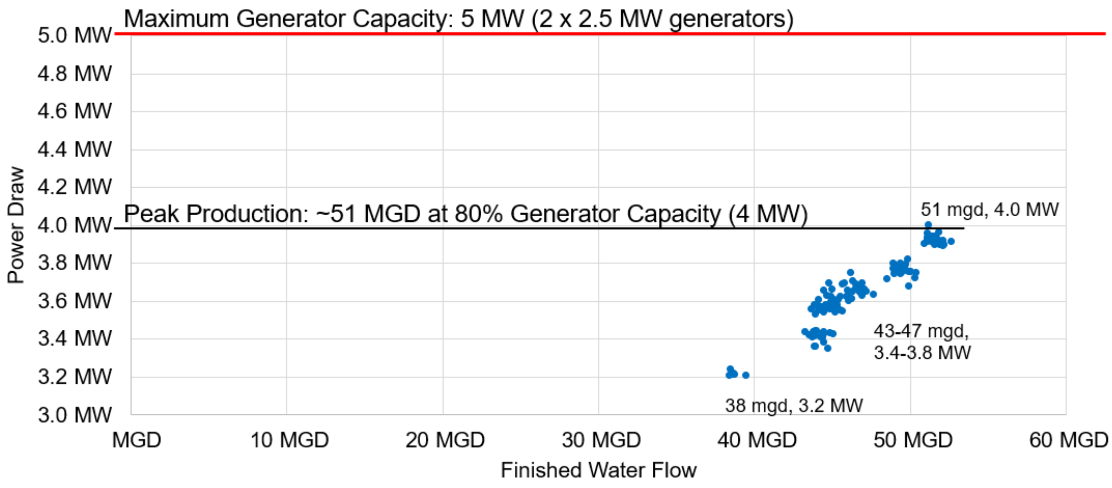


Figure 7.8 Plant Production and Corresponding Power Draws for September 2020 Operation on Backup Generator Power

Peak production while on generator power was approximately 51 mgd at 80 percent load (4 MW) on the generators. Peak generator capacity is driven by peak electrical loads during equipment startup, and maximum running loads for continuous operation are typically around the 80 percent load observed for September 2020 operation.

Manufacturer product data for OSHG systems was reviewed to determine the power requirements. For the range of plant flow rates on generator power and typical chemical doses, no more than two 1,000 lb/day generators would need to be online. Peak power draw for each 1,000 lb/day generator is approximately 0.1 MW, so maximum power draw for the full OSHG system would be approximately 0.2 MW.

Assuming maximum generator running load remains limited to 80 percent or 4 MW, this additional load of 0.2 MW needed will reduce maximum production capacity on generator power. With 3.8 MW of load on the generators for existing equipment, plant flow rates were between 48 and 50 mgd. Based on this limited evaluation of historical operating data, operating the OSHG system while on generator power may reduce maximum plant production capacity to 45 to 50 mgd. While further evaluation will be needed to determine how to integrate OSHG into the existing plant power system, there are no apparent concerns producing the design capacity of 37.5 mgd while on generator power.

7.2.7.4 Building Siting

JWC's most recent facility plan for the WTP, the 2018 Facility Plan, laid out phased construction of new infrastructure to replace existing facilities to meet seismic resilience goals and add new facilities needed to meet ultimate build-out plant capacity requirements. The facility plan identified a location for a new chemical building, shown in Figure 7.9, that would consolidate all chemical facilities into a single location and include an on-site hypochlorite generation system. The footprint allocated for the building is approximately 90 feet by 160 feet (14,400 square feet) and is located in the current footprint of the abandoned gravity thickeners behind the current main operations building.

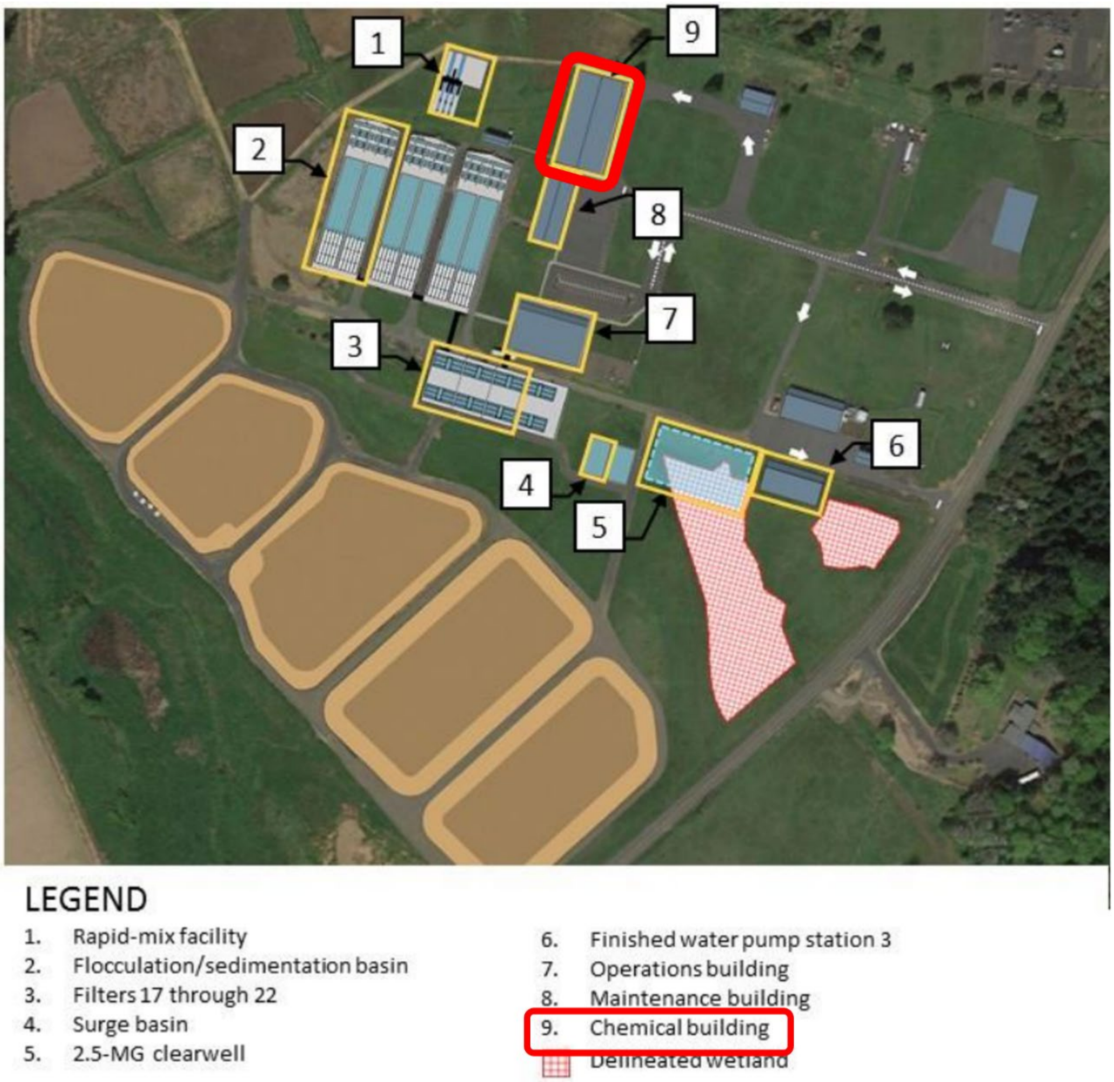


Figure 7.9 JWC WTP Future Site Layout (adapted from 2018 *Water Treatment Facility Plan [CH2M]*)

Given the chlorine supply issues and significant safety concerns with the chlorine gas system, JWC intends to accelerate the construction of a new disinfection alternative ahead of the original implementation order in the 2018 Facility Plan. Constructing the disinfection facility at the location shown in Figure 7.9 poses challenges for construction and operation in the near-term. The main concerns with the location are the proximity to the current operations building and lack of access from existing driveways through the plant site. The ability to provide easy drive-through access for chemical deliveries within the constraints of existing roads and infrastructure is limited. Additionally, work on demolishing one or both abandoned gravity thickeners would need to be accelerated to accommodate a new disinfection building at that site.

JWC staff identified alternative locations on the WTP site that may be able to accommodate a new disinfection building. These are shown in Figure 7.10. These locations would be accessible with existing roads, were not otherwise allocated for use in the 2018 Facility Plan, and appear to have enough space to house the chemical building footprint proposed in the 2018 Facility Plan. It is assumed that the proposed locations would only house the facilities needed for the selected disinfection alternative in the near-term with facilities for all other chemicals to be constructed later.

Figure 7.10 Potential Alternative Locations for Near-Term Disinfection Building and Future Chemical Building (Image Source: Google Earth)

Additional siting evaluations are needed to assess the viability of these locations. Both locations are further away from the future treatment facilities than the chemical building location proposed in the 2018 Facility Plan. This will result in longer pipelines to reach treatment facilities and a less congruous future WTP layout. Further study is also needed to understand potential existing and future conflicts with buried infrastructure in these locations. Finally, the ultimate chemical building footprint should be revisited within the context of the chemical storage criteria established for this disinfection alternatives evaluation. For the alternatives presented in this plan, chemical storage for the disinfection alternatives was sized to maintain the desired number of days of storage assuming tanks were 80 percent full. This standard increases the required chemical storage beyond typical design criteria, which are typically based on providing the desired number of days of chemical storage assuming all tanks are 100 percent full. Establishing the 80 percent full criteria for all chemicals on the JWC site may result in a larger required footprint for the overall chemical building than shown in the 2018 Facility Plan.

7.2.8 Economic Evaluation

7.2.8.1 Cost Estimating Methodology

Comparative cost estimates were prepared to evaluate the economic impacts of each of the disinfection improvement alternatives. These cost estimates are comparative in nature and are equivalent to an Association for the Advancement of Cost Engineering (AACE) Class 5 estimate, used for "order-of-magnitude" cost comparison, with an expected accuracy range between minus 20 to minus 50 percent on the low end and plus 30 to plus 100 percent on the high end. All costs presented are in June 2022 dollars and were developed using historical project data, project allowances, and vendor quotes.

Three cost estimates were developed for each alternative:

- Capital Cost.
- O&M.
- Net Present Value (NPV).

Capital Costs

Direct construction capital cost estimates focused on the major facilities and equipment required with additional allowances for mechanical and electrical requirements. Total construction cost estimates also included general percentage factors that were applied to direct costs to estimate a total construction cost:

- General Conditions: 12 percent.
- Escalation: 10 percent per year.
- Scope Contingency: 30 percent.
- Contractor Overhead and Profit: 10 percent.

Escalation costs were included at a rate of 10 percent per year based on recent market trends in construction costs. Capital costs were estimated in June 2022 dollars and escalated forward two years to account for an early likely construction timeline. The scope contingency included accounts for unknowns and items not specifically included in the estimate for this analysis.

An additional 10 percent cost factor was applied to the total construction cost for engineering design services, with an additional two percent cost factor for engineering services during construction. Additional lump sum cost estimates were included for administration, permitting, and third-party construction management costs to develop the total project cost estimate presented for each alternative. See Appendix K for cost breakdowns.

O&M Costs

O&M cost estimates focused on the following three major categories to differentiate between the alternatives, with additional details for each category as follows:

- Chemical Usage:
  - Annual chemical usage was estimated using average production and average chlorine doses.
  - To estimate salt usage for OSHG, it was assumed that three pounds of salt are required for each pound of chlorine generated (based on typical manufacturer values).
  - Chemical costs were based on recent quotes provided for the JWC WTP and other regional WTPs:
    - Bulk sodium hypochlorite delivery: $1.60 per pound of chlorine.
    - Bulk salt delivery: $0.13 per pound.
- Power Usage:
  - Power usage was estimated from major equipment motor loads.
  - For the OSHG system, a power requirement of 2.5-kilowatt hour (kWh) per pound of chlorine generated was assumed based on typical manufacturer values.
  - Average power cost of $0.075 per kWh was based on past utility bills for the JWC WTP.
- Equipment Maintenance:
  - Maintenance costs were estimated as two percent of total replacement asset value for major equipment in each alternative.

Note, Oregon Law (Oregon Revised Statue 453.402) requires facilities that house hazardous substances pay a fee to the State Fire Marshal based on the chemical hazard level. While multiple water treatment chemicals at the JWC WTP are classified as hazardous chemicals, JWC is exempt from paying the fees because the chemicals are necessary for providing a public service supply drinking water. No costs for these fees were included in this analysis.

O&M costs were developed to a level sufficient to capture the most significant annual expenditures and identify major differentiators among alternatives for comparison and are not intended to capture comprehensive operations costs. O&M costs were also broken down into fixed and variable costs. Fixed costs include equipment maintenance only, while variable costs include power and chemical usage, with costs depending on average production.

For the bulk hypochlorite and full OSHG alternative, annual chemical and power costs were estimated based on average annual flow rate and average chemical doses. For the bulk hypochlorite with backup OSHG alternative, annual O&M costs depend on how often the OSHG is used. For this analysis, it was assumed the backup OSHG system would operate for 30 days per year with plant production equal to annual average winter day demands. Chlorine dose while using the OSHG system was assumed to be equal to the annual average. This frequency of operation would ensure full turnover of the 30 days of salt storage annually, at a minimum. For the remainder of the year the bulk hypochlorite system would be used with production equal to the average day demand accounting for the 30 days of operating the OSHG system at winter average day demands.

See Appendix K for O&M cost breakdowns for each alternative.

NPV Costs

NPV costs were calculated for 20 years of operation (2022 to 2042) to account for the differences in annual operating costs among the alternatives. For the NPV costs, a real rate of 2.35 percent was used to discount future operating costs to June 2022 dollars. Average annual production was scaled linearly from the current 32 mgd to the forecasted 2042 production of 42 mgd to account for variable chemical and power costs that depend on production volumes.

7.2.8.2 Comparative Cost Evaluation

Table 7.8 compares the economic factors differentiating the alternatives, including the capital costs, annual O&M costs, cost per pound of chlorine, and the 20-year NPV. Figure 7.11 also summarizes the results of the comparative cost evaluation graphically for the alternatives.

Table 7.8 Summary of Capital, O&M, and NPV Costs for Each Alternative

Alternative	Capital Costs(1)(2)(8)	Annual O&M Cost(1)(3)(8)	Cost per Pound of Chlorine(4)	20-Year NPV(5)
Bulk Hypochlorite	$8.9 million	$358,000	$1.68	$16.1 million
Full OSHG	$14.6 million	$184,000	$0.86	$17.6 million
Bulk with Backup OSHG(6)	$11.5 million	$366,000	$1.71	$18.4 million
Bulk Hypochlorite Future OSHG(7)	$9.1 million	$358,000	$1.68	$16.3 million

Notes:
(1) Capital costs estimated in June 2022 dollars. Costs escalated forward at a rate of 10 percent per year for two years.
(2) Expected cost accuracy range +50 percent to -30 percent.
(3) Estimated annual cost to produce current 31.9 mgd average day demand.
(4) Based on average annual O&M cost and annual chlorine usage at average dose of 2.2 mg/L and current 31.9 mgd average day demand.
(5) NPV costs include the initial capital investment as well as fixed O&M costs, accrued each year, and increasing variable O&M costs based on projected average annual future flow rates through the planning period (20 years).
(6) For annual O&M costs, on-site hypochlorite generation system assumed to operate for approximately 30 days per year to produce winter average day demand. System operated to fully turn over 30 days of salt storage once per year.
(7) For annual O&M costs and 20-year NPV, it was assumed that bulk hypochlorite was used for the entire 20 years.
(8) See Appendix K for capital cost and O&M cost breakdowns.

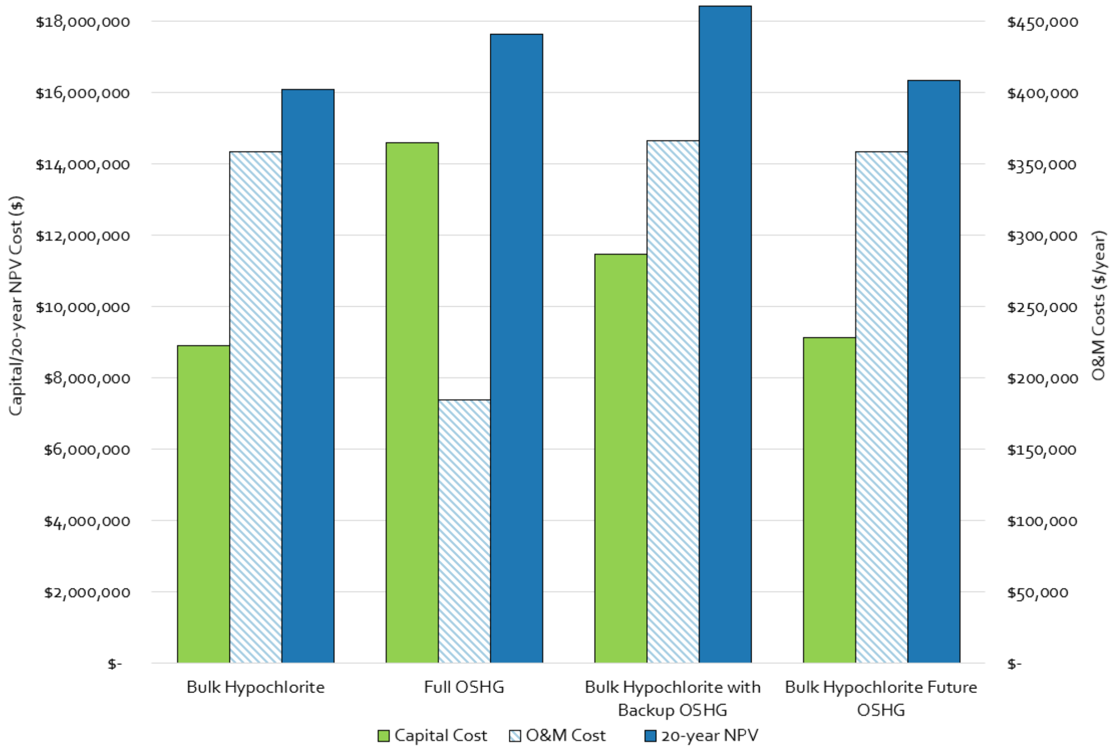


Figure 7.11 Summary of Capital, O&M, and NPV Costs for Each Alternative

As shown in Table 7.8 and Figure 7.11, bulk hypochlorite and bulk hypochlorite with provisions for future OSHG have the lowest capital cost and 20-year NPV even with the high annual O&M costs for chemical delivery. Bulk with provisions for future OSHG has a $0.2 million higher capital cost, and 20-year NPV, that is driven by the larger building footprint to accommodate full OSHG in the future.

Full OSHG capital costs are more than 60 percent higher than the bulk hypochlorite alternatives predominantly due to the higher costs for the OSHG equipment. The 50 percent reduction in O&M costs relative to bulk hypochlorite are not substantial enough to offset the difference in capital cost for the 20-year NPV. While full OSHG has a higher NPV, 20-year NPV costs are generally comparable given the wide accuracy range for Class 5 cost estimates.

Bulk hypochlorite with backup OSHG has a capital cost between the bulk hypochlorite alternatives and full OSHG, with the cost savings mainly coming from the smaller size of the generation system. Annual O&M costs are comparable to bulk hypochlorite. O&M costs presented here assume 11 months of operation with bulk hypochlorite. In general, savings for operating the OSHG system one month per year are offset by the increased maintenance costs for the OSHG equipment. This is reflected in the highest cost per pound of chlorine, comparable to that of bulk hypochlorite. If the backup OSHG system were to be used more frequently, the cost per pound of chlorine would decrease and trend closer to the cost for full OSHG. Overall, the high annual operating costs and higher capital cost relative to bulk hypochlorite result in the highest 20-year NPV cost.

7.2.9 Non-Economic Evaluation

In addition to the economic analysis, a non-economic analysis was used to further evaluate the four disinfection alternatives. Non-economic criteria were developed with input from JWC WTP staff members and JWC partners. Relative weights were assigned to each criterion through a collaborative process to reflect JWC's priorities, with weights ranging from 1 (lowest importance) to 5 (highest importance). Table 7.9 summarizes the non-economic criteria and weightings developed for this analysis.

Table 7.9 Non-Economic Criteria and Weighting

Criteria	Weight	Description
Safety(1)	4	• Risks and potential effects on the health and safety of WTP staff during everyday operation and maintenance. • Risks and potential effects to the health and safety to civilians and communities in the vicinity of the JWC WTP and along the chemical delivery route to the facility. • Minimizes security concerns associated with disinfection system.
Resilience & Reliability	5	• Provides a reliable supply of chemical to keep the system operational in the event of chemical supply disruptions. • Technology is proven and well-vetted in the industry. • Resilient to emergency disruptions such as earthquakes and flooding. • Ability to provide standby or backup capabilities if primary equipment fails or is taken offline for service or maintenance.
Water Quality	3	• Minimizes effects on finished water quality and disinfection byproduct formation. • Minimizes treatment process impacts.
Environmental / Land Use	2	• Minimizes potential harm to the environment surrounding the JWC WTP. • Maximizes energy efficiency and efficient use of other resources (chemicals, transportation, etc.) during construction and operation. • Minimizes challenges and/or risk associated with permit acquisitions.
Constructability	3	• Ease and efficiency of construction and implementation. • Footprint preserves available space on site for future expansion. Ease of expansion with future WTP expansions. • Ease of integration with existing WTP infrastructure both during construction and long-term operations. • Minimizes potential for conflicts with existing equipment and/or structures. • Maintains flexibility for expanding facilities in the future.
O&M	3	• Ease of operation and maintenance. • Minimizes time required to comply with safety regulations.

Notes:

(1) Safety considerations related to the existing chlorine gas were the trigger for this disinfection alternatives analysis. Safety remains a top priority for JWC but was not weighted as a 5 as all options considered provide safer alternatives than chlorine gas.

Each alternative was evaluated and given a score between 1 (worst meets criterion) to 5 (best meets criterion). The assigned scores were multiplied by the corresponding weighting factors to determine a score for each criterion and weighted overall score for each alternative. Figure 7.12 summarizes the results of the non-economic evaluation; a complete breakdown of the scoring and associated reasoning is provided in Appendix K.

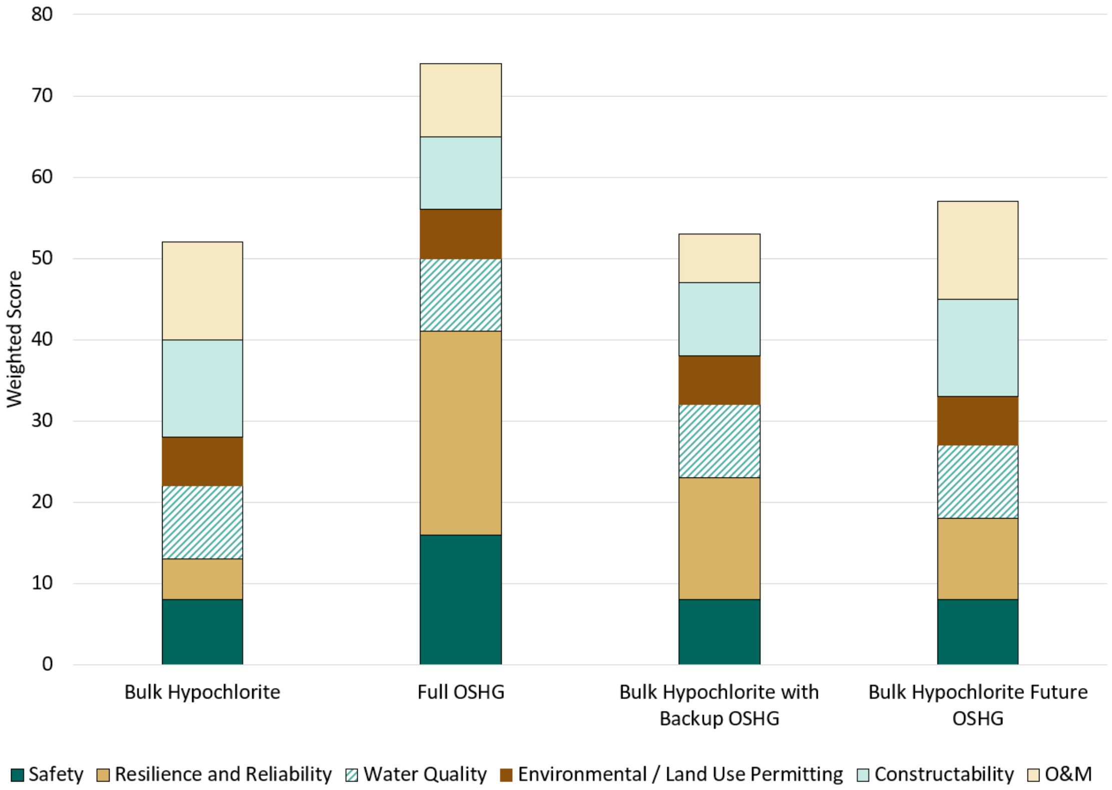


Figure 7.12 Disinfection Alternatives Non-Economic Scores

As shown in Figure 7.12, the full OSHG alternative scores the highest because it scores highly in the most important criteria, Safety and Resilience & Reliability. OSHG poses few health and safety risks to WTP staff and the surrounding community and does not rely on corrosive bulk hypochlorite like the other two alternatives. In terms of Resilience & Reliability, full OSHG provides multiple benefits. Salt is a non-hazardous chemical that can be more easily supplied in the event a local supply outage occurs, while bulk hypochlorite remains vulnerable to supply disruptions like those experienced by JWC in 2021. Additionally, the tank storage can be used for bulk delivery as a backup supply source; the fully sized OSHG system provides JWC with a fully redundant system that can meet peak demands using generated hypochlorite or bulk delivery.

The bulk hypochlorite with provisions for future OSHG scores slightly higher than the bulk hypochlorite alternative for Resilience & Reliability because planning for the future OSHG sets the stage for JWC to convert to a more resilient source of sodium hypochlorite in the future.

All four alternatives score similarly with respect to Water Quality and Environmental/Land-use permitting. No significant effects on finished water quality are anticipated, but JWC should secure high-quality chemicals to help manage disinfection byproduct levels (bromate for OSHG and chlorite/chlorate for bulk hypochlorite). No major permitting challenges are anticipated for any of the alternatives.

Bulk hypochlorite and bulk hypochlorite with provision for future OSHG score the highest for Constructability and O&M categories while the bulk hypochlorite with backup OSHG system scores the lowest. Bulk hypochlorite requires the least amount of equipment to maintain. The full OSHG and bulk hypochlorite with backup OSHG systems have similar footprints and Constructability scores. The bulk hypochlorite with backup OSHG alternative scores lowest for O&M because it will be the most complex system to operate and maintain. Switching between 12.5 percent and 0.8 percent hypochlorite adds operational complexity and increased maintenance will be needed to ensure the generation equipment remains functional for idle periods when not in use.

7.2.10 Recommendations

Based on the non-economic evaluations, and discussion with JWC WTP staff and JWC partners, full OSHG was determined as the most beneficial alternative for the long-term. However, given the higher upfront capital costs and challenging bidding and construction environment at the time of this Plan, bulk hypochlorite with provisions for future conversion to OSHG was selected as the recommended disinfection alternative. This provides JWC the space and flexibility to transition to full OSHG in the future. Key factors leading to the selection of this alternative include the following:

- Fewer safety concerns with bulk sodium hypochlorite compared to chlorine gas.
- Lower capital cost ($9.1 million versus $14.6 million for full OSHG).

Long-term, full OSHG is the preferred alternative for the following reasons:

- Increased resilience and reliability for the full OSHG system. The OSHG system can fully meet all treatment process needs with generated hypochlorite or through delivered bulk hypochlorite in the event of an extended catastrophic equipment failure. Salt can more easily be supplied from long distances and is likely less susceptible to supply disruptions than bulk hypochlorite.
- Reduced price volatility compared to chlorine gas and bulk hypochlorite such as observed during the 2021 supply disruptions.
- Lower annual operating costs compared to bulk hypochlorite.

Figure 7.13 shows a conceptual layout for the selected bulk hypochlorite alternative with flexibility to transition to full OSHG in the future. Total estimated capital cost for the selected alternative is $9.1 million (estimated cost does not include OSHG equipment or salt storage slabs).

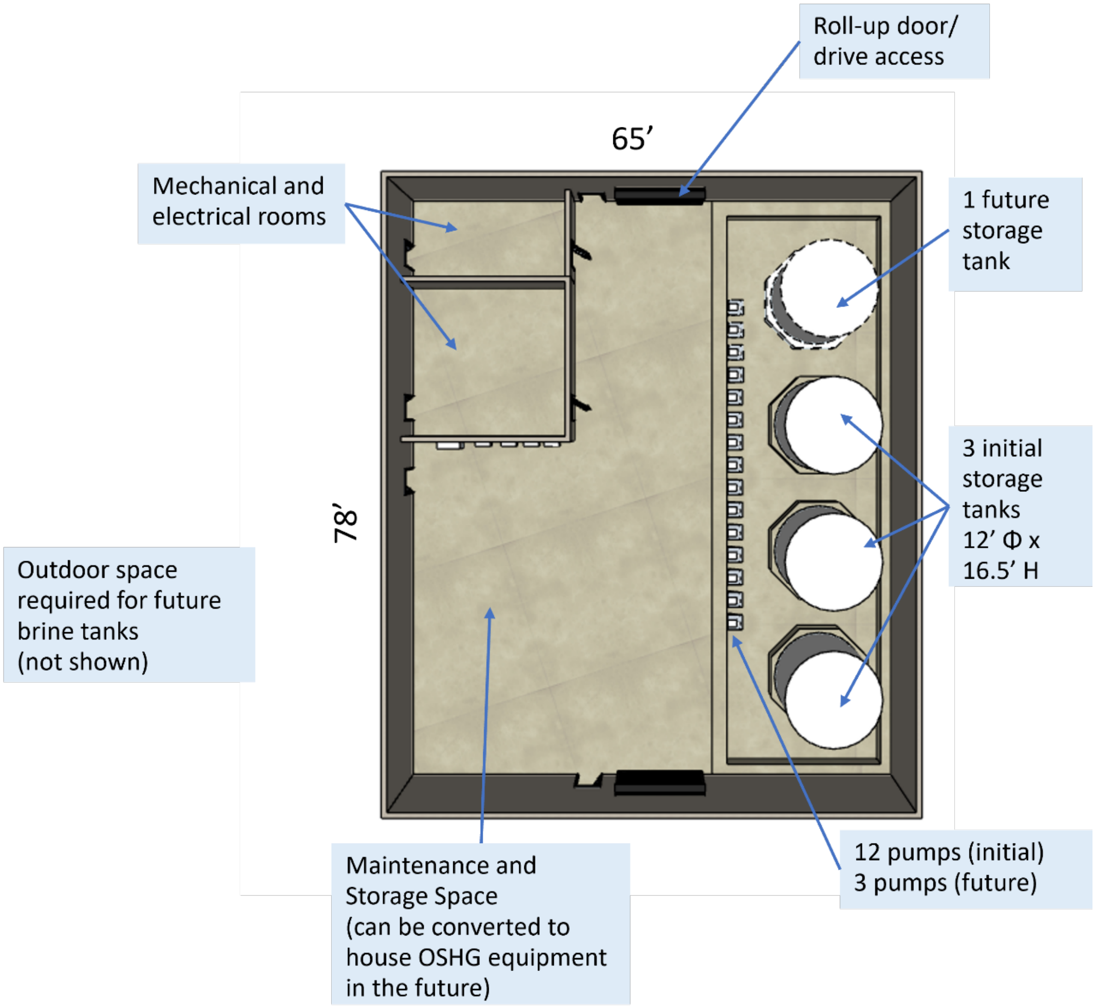


Figure 7.13 Recommended Disinfection Alternative - Bulk Hypochlorite with Provisions for Future Conversion to Full OSHG

7.3 JWC WTP Improvement Plan

The section provides a comprehensive list of WTP improvement projects identified from previous JWC planning efforts and studies as well as the evaluations completed in this master plan for incorporation into the capital improvement program (CIP) presented in Chapter 9.

7.3.1 Previous Evaluations Reviewed

Recommended CIP projects were taken from several sources, including the following:

- *Technical Memorandum No. 1 Capital Improvement Program Update* (2015 CIP Update).
- *Water Treatment Facility Plan* (2018 Facility Plan):
  - Projects primarily consist of capital improvement maintenance projects (CIMP) for 2020 through 2026 and phase II seismic replacement projects.
- *Finished Water Pump Station 1, Pump and Motor Replacement Preliminary Design Report*.
- *2021 Supervisory Control and Data Acquisition (SCADA) Master Plan*.
- Chapter 2 and Technical Memorandum (TM) 03 of this master plan.
- Disinfection alternatives analysis discussed in Section 7.2.

Projects identified from these documents were reviewed and cross-checked to omit duplicates and projects that have been completed.

7.3.2 WTP Improvement Projects

WTP improvement projects were grouped into three categories based on the overall project purpose:

- **Operations and Maintenance:** Projects that address maintenance of existing facilities and address operational challenges and limitations. These include projects identified as 2020-2026 CIMP in the 2018 Facility Plan and additional projects identified during development of this plan based on discussions with JWC WTP staff members. Any additional studies related to operation of the JWC WTP are included in this category. These projects and descriptions are shown in Table 7.10.
- **Seismic and Resilience Improvements**: Projects identified in the 2018 Facility Plan as part of the 2026 Seismic Resiliency Project that cover seismic upgrades identified in the 2015 CIP Update. These projects include seismic upgrades beyond life safety improvements or full replacement of existing facilities to ensure a seismically resilient 85 mgd capacity. Projects also include all projects identified in the 2021 *SCADA Master Plan* to improve resilience of the JWC's SCADA system. Table 7.11 provides details for all seismic and resilience improvement projects.
- **WTP Expansion:** Projects identified in the 2018 Facility Plan to expand peak capacity beyond 85 mgd or add additional treatment processes. Note, based on the demand projections developed in Chapter 5, maximum day demands are not anticipated to exceed the current peak capacity of 85 mgd within the 20-year planning horizon for this plan. WTP expansion project implementation timelines and other details should be revisited as demands increase and the need for additional capacity arises. Expansion projects are summarized in Table 7.12.

Table 7.10 Operations and Maintenance Projects

Project Description	Facility	Source Document
Add additional PRS station (at JWC WTP or Evergreen Reservoir) to assess impacts of JWC WTP pH adjustment on pipe scale composition and corrosion.	Study - Corrosion Control	TM 03 - Future Water Quality Blending Evaluation (see Appendix G)
Add hatches to floc/sed Basins A-C. (CIMP 2020 - 2016).	Floc/Sed Basins	2018 Facility Plan / 2015 CIP Update
Replace finished water pump 7.	Finished Water Pump Station 2	2018 Facility Plan / 2015 CIP Update (2020-2026 CIMP)
Replace finished water pump 8 and backwash pump 3.	Finished Water Pump Station 2	2018 Facility Plan / 2015 CIP Update (2020-2026 CIMP)
Replace finished water pump 9 and backwash pump 4.	Finished Water Pump Station 2	2018 Facility Plan / 2015 CIP Update (2020-2026 CIMP)
Demolish gravity thickeners.	Solids Handling	Chapter 2
Replace MCC MCPA and MCC-8 in Spring Hill Pump Plant. Replace MCCA MCS and MCO in Ops Building. Replace MCC MPA and Panel LGG in FWPS 1.	Electrical	2018 Facility Plan / 2015 CIP Update (2020-2026 CIMP) TM 05 - Electrical System Condition Assessment (see Appendix I).
Update JWC CIP Master Plan every 5 years.	General	2015 CIP Update

Note:
Abbreviations: floc/sed - flocculation and sedimentation; MCC - motor control center; FWPS – finished water pump station.

Table 7.11 Seismic and Resilience Improvement Projects

Project Description	Facility	Source
Replace rapid mix facility with jet injection or inline mixing facility.	Rapid Mix	2018 Facility Plan
New rapid mix facility yard piping from new rapid mix facility to floc/sed basins.	Rapid Mix	2018 Facility Plan
Replace floc/sed Basins A-C with new floc/sed basins to provide 85 mgd total capacity.	Floc/Sed Basins	2018 Facility Plan
Strengthen Basins D-G baffles for improved structural seismic resiliency.	Floc/Sed Basins	2018 Facility Plan / 2015 CIP Update
Replace Filters 1-14 with new filters to provide 85 mgd of capacity.	Filters	2018 Facility Plan
Replace existing clearwell with a new 2.5 MG clearwell, including foundation stabilization for the new clearwell.	Clearwell	2018 Facility Plan / 2015 CIP Update
Replace finished water pump station 1 with new finished water pump station 3.	Finished Water Pump Stations / Backwash Pump Stations	2018 Facility Plan
Finished water pump station 2 seismic improvements.	Finished Water Pump Stations / Backwash Pump Stations	2018 Facility Plan / 2015 CIP Update
Second washwater surge basin.	Solids Handling	2018 Facility Plan
New chemical building with alum, polymer, and powdered activated carbon storage and chemical metering.	Chemicals	2018 Facility Plan
New bulk hypochlorite disinfection facility (designed for future conversion to OSHG).	Chemicals	Section 6.2
New operations and maintenance building.	Operations	2018 Facility Plan
Hydropneumatic surge tank liquefaction mitigation.	General	2018 Facility Plan / 2015 CIP Update
Core SCADA Improvements (FP-1)	SCADA	2021 SCADA Master Plan
Policies and Procedures Development (FP-2)	SCADA	2021 SCADA Master Plan
Physical Security and Access Control Improvements (FP-3)	SCADA	2021 SCADA Master Plan
Governance Initiative (FP-4)	SCADA	2021 SCADA Master Plan
Asset Management Inventory (FP-5)	SCADA	2021 SCADA Master Plan
Message Queuing Telemetry Transport Pilot and Implementation (FP-6)	SCADA	2021 SCADA Master Plan
Fiber Optic Improvements (FP-7)	SCADA	2021 SCADA Master Plan
Earthquake Early Warning System (FP-8)	SCADA	2021 SCADA Master Plan
Day without SCADA Exercise (FP-9)	SCADA	2021 SCADA Master Plan
Consequence-Driven-Cyber-Informed Engineering Training (FP-10)	SCADA	2021 SCADA Master Plan
SCADA Master Plan Update (FP-11)	SCADA	2021 SCADA Master Plan

Note:
Abbreviations: MG - million gallons.

Table 7.12 WTP Expansion Projects

Project Description	Facility	Source Document
Construct new filters.	Filters	2018 Facility Plan
Construct new floc/sed basins or add additional plate settlers to increase floc/sed capacity.	Floc/Sed Basins	2018 Facility Plan
Construct new advanced treatment.	–	2018 Facility Plan
Construct new solids drying beds.	Solids Handling	2018 Facility Plan

7.4 Siting of Future JWC WTP Facilities

This section provides an overview of the siting study completed as part of this planning effort. The complete siting study evaluation is provided in Appendix O.

JWC's 2018 Facility Plan, established the basis and vision for future expansions and improvements at the JWC WTP, including site layouts and project sequencing. For this planning effort, projects identified in the WTP improvement plan, Section 7.3, were aggregated, prioritized, and assigned to CIP implementation phases during development of the CIP, presented in Chapter 9. CIP projects were either assigned a project timing within the 20-year CIP or a 20-year period beyond the CIP for additional seismic resilience projects. This CIP development effort changed the sequencing and prioritization of JWC WTP projects relative to what was established in the 2018 Facility Plan. The objective of the siting study was to develop an updated site layout and vision for the JWC WTP that accounts for all current CIP projects and follows the timing and prioritization identified in Chapter 9.

For the siting study, three representative site layouts were developed in collaboration with JWC staff and then presented to JWC staff and JWC partners for selection of a preferred site layout alternative. Figure 7.14 shows the selected site layout alternative after expansion to 105 mgd. See Appendix O for additional detail on the development of the site layout alternatives.

The preferred site layout, shown in Figure 7.14, includes several potential adjustments that could be considered during future phasing (shown with dashed boxes and callouts). These additional considerations incorporated elements from the other site layout alternatives that could be implemented without affecting the overall vision for the JWC WTP, and consist of the following:

- Locating future floc/sed basins within the footprint of existing floc/sed basins A - C. Note, further feasibility evaluations will be necessary to determine if the plant can operate at reduced capacity and if it is possible to demolish a portion of basins A - C while keeping the remaining basins in service.
- Orienting the clearwell and FWPS as shown in either layout 1 or layout 2, which are shown in Appendix O.
- Locating the operations building closer to the front of the plant and swapping its location with the maintenance building.

As design of future WTP CIP projects progresses, it is recommended that JWC review the constructability of future facilities with their selected contractor(s) and adjust the planned future site layout as required.

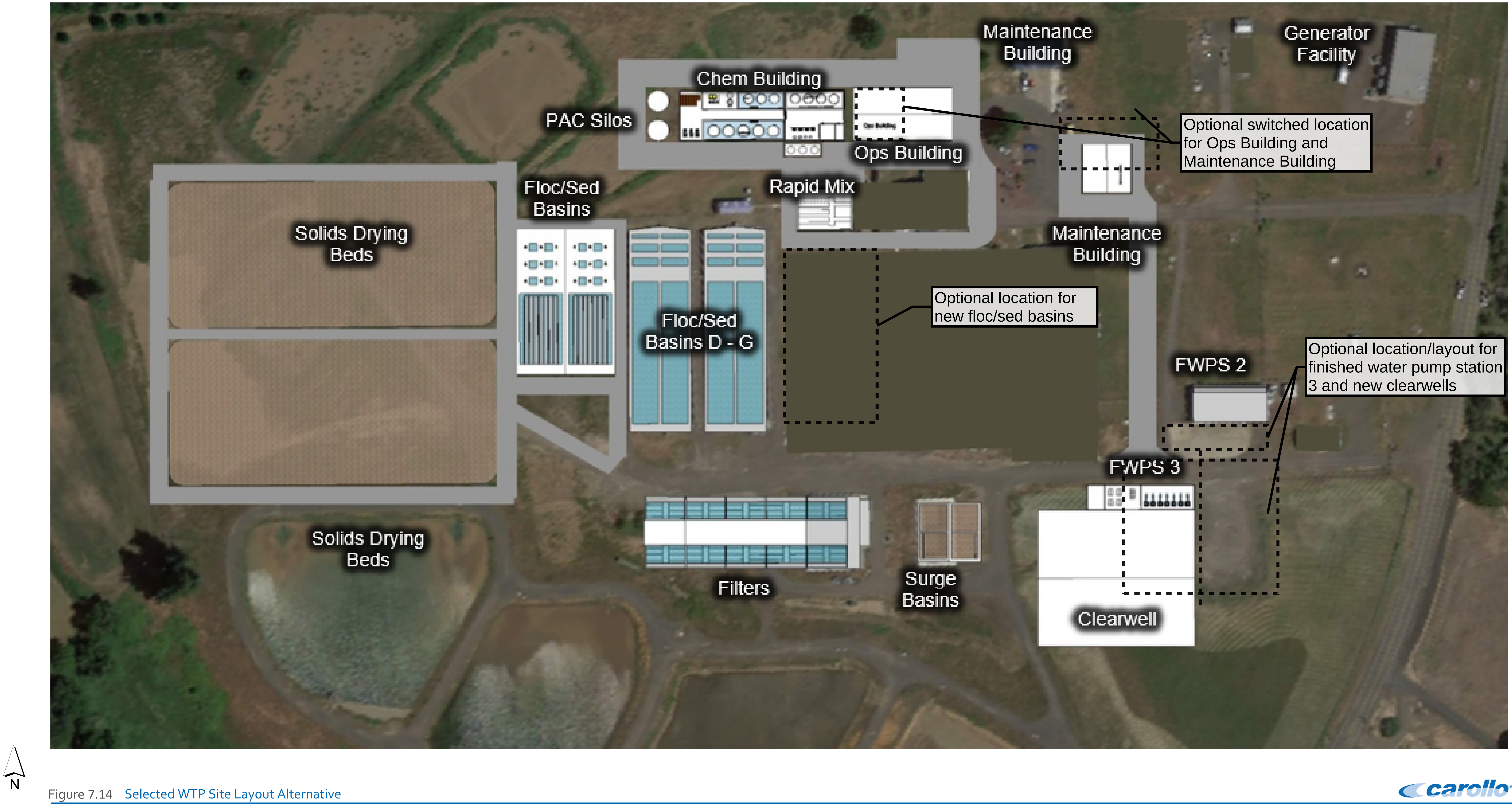


Figure 7.14 Selected WTP Site Layout Alternative

Chapter 8

TRANSMISSION SYSTEM EVALUATION

8.1 Introduction

The Joint Water Commission (JWC) owns and operates a transmission system that includes the Fern Hill reservoirs and the following three major transmission lines that serve 450,000 customers:

- The south transmission line (STL): approximately 64,000 linear feet of 45- and 42-inch-diameter, bar-wrapped concrete cylinder pipelines (CCPs).
- The north transmission line (NTL): approximately 74,200 linear feet of 72- and 66-inch-diameter, mortar-lined and coated steel pipelines (MLCS).
- The 24-inch-diameter Forest Grove-Hillsboro (FGHB) transmission main: approximately 8,200 linear feet of 24-inch-diameter CCP piping that serves the City of Forest Grove as well as the City of Hillsboro's Upper System.

In addition to these major lines, the 42-inch-diameter CCP STL-NTL north-south intertie (N-S Intertie), measures approximately 7,700 linear feet.

This chapter presents an analysis of JWC's storage capacity and transmission capacity as well as results from the surge analysis, as outlined in Appendix H. The chapter also summarizes recommended improvements from the *2020 Update to JWC's Cathodic Protection Program* (2020 Cathodic Study), *Technical Memorandum (TM) 1: Condition Assessment*, and *TM 2: Transmission System Seismic Resiliency Evaluation*. Additional infrastructure improvements identified by JWC are also included in this chapter.

8.2 Storage Capacity Analysis

This section summarizes the storage capacity analysis, which compares JWC's existing storage to the storage analysis criteria.

8.2.1 Existing Storage

JWC has two finished water reservoirs, the Fern Hill reservoirs, each with a capacity of 20 million gallons (MG), with a total storage capacity of 40 MG. JWC operates the Fern Hill reservoirs to maximize partner storage, meet the WTP's operating requirements, and respond to emergencies. Storage in the Fern Hill reservoirs is divided into three components, each with different volumes of water:

- Operational storage: 20 percent (8 MG).
- Emergency storage: 20 percent (8 MG).
- Partner storage: 60 percent (24 MG).

Under normal operations, reservoir storage is typically maintained above 60 percent full (approximately 26 MG of storage).

8.2.2 Methodology

To analyze JWC's storage capacity, the volume of existing operational storage was compared to the volume of storage required to meet daily fluctuations in demand. A diurnal curve was developed using hourly supervisory control and data acquisition (SCADA) data from partner connection flows for the maximum demand week from 2021 (August 1st – 7th). Figure 8.1 shows the average flow for each hour during the maximum demand week when the average daily demand was approximately 56 million gallons per day (mgd).

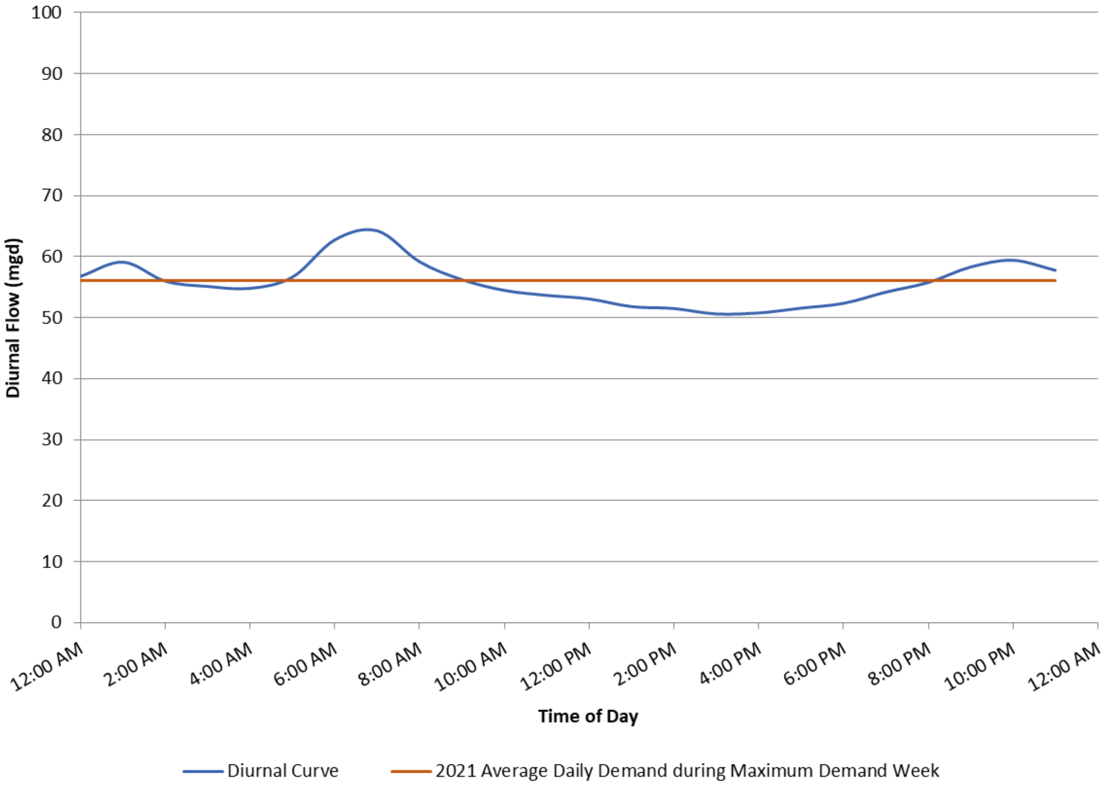


Figure 8.1 Diurnal Curve from Maximum Demand Week in 2021

This diurnal curve was then normalized to show the diurnal demand coefficient, where a coefficient of 1 is equal to the average daily demand, as shown in Figure 8.2.

This diurnal curve was used to determine the volume of storage needed to meet fluctuations in demand above the daily average flow, which is represented by the blue shaded area above the 1.0 diurnal demand coefficient but below the diurnal curve.

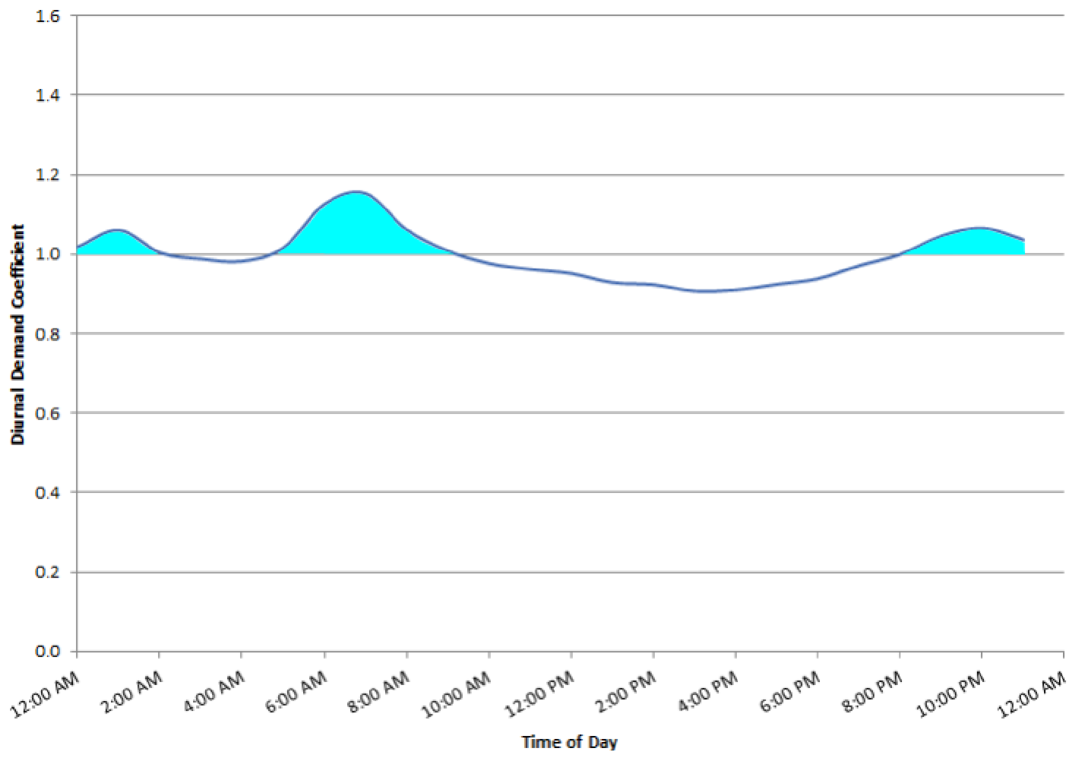


Figure 8.2 Diurnal Demand Coefficient Curve

8.2.3 Results

Table 8.1 summarizes the storage required to meet daily fluctuations in demand for the existing (2021) scenario, 10-year (2032) scenario, and 20-year (2042) scenario.

Table 8.1 Storage Requirement to Meet Daily Demand Fluctuations

Year	Total MDD Projection (mgd)[(1)]	Storage Required to Meet Daily Demand Fluctuations (MG)
2021	72.5	1.9
2032	77.4	2.0
2042	82.8	2.1

Note:
Abbreviation: MDD - maximum day demand.
(1) Total MDD as shown in Table 5.6.

As shown in Table 8.1, the storage required to meet daily demand fluctuations for all planning years is well below JWC's existing operational storage capacity of 8 MG. Therefore, no storage improvements are recommended.

8.2.4 Recommendations

Based on the comparison of JWC's existing operational storage to the storage required to meet daily demand fluctuations, no storage improvements are recommended.

Note that in addition to storage at the Fern Hill reservoirs, JWC's policy, as outlined in *Chapter 3 Planning Considerations* (Section 3.1.2.1 Storage), is that each partner must maintain and operate their own storage facilities and/or be able to utilize an alternative water source capable of meeting three days of average demands. Partners can utilize capacity from aquifer storage and recovery (ASR) facilities and regularly maintained wells to meet the alternative water source requirement, provided that the ASR pumping capacity has a backup power supply.

8.3 Transmission System Capacity Analysis

This section describes the transmission system capacity analysis, which compares the results produced from JWC's updated and calibrated model to established evaluation criteria.

8.3.1 Methodology

JWC's existing updated and calibrated hydraulic model, received in March 2021, was updated as described below and then used to evaluate the transmission system capacity. The hydraulic model is in Innovyze's InfoWater Pro software and includes both JWC and City of Hillsboro infrastructure. For the transmission system capacity evaluation, only JWC infrastructure was active in the hydraulic model.

Partner demands were allocated in the hydraulic model based on SCADA data and demand projections as outlined in Chapter 5 Water Demand Forecast and Supply Strategy. The model was run as a steady-state model for each demand scenario, and the results were compared with established evaluation criteria. Deficiencies were identified, and improvements are recommended as needed.

Additionally, the total and firm capacities of the finished water pump stations were reviewed in the model and compared to the total capacity of the WTP.

8.3.1.1 Evaluation Criteria

Chapter 3 outlines the planning criteria for JWC's transmission system that were used to evaluate the results from the hydraulic model. These criteria include minimum hydraulic grade line (HGL) requirements and a maximum transmission main velocity.

JWC supplies water at the necessary HGL to operate the control valves at each partner interconnection. JWC has two minimum HGL requirements, one for the end of the NTL and one for the end of the STL, summarized in Table 8.2. Provided JWC meets the minimum HGL at these locations, the HGL at all other partner interconnections will be sufficient to meet the needs of the partners. As noted in *Chapter 3 Planning Considerations* Section 3.1.2.3, transmission main velocity shall not exceed 10 feet per second (ft/s).

Note, the contractual minimum HGL elevation is based on the City of Hillsboro's revised benchmark elevation of 194.22 feet (The JWC transmission main design was originally based on NGVD29 Washington County Benchmark elevation of 194.31 feet).

Table 8.2 JWC Partner Minimum HGL Requirements

Location	Minimum HGL (feet)(1)
End of NTL – TVWD interconnection	450
End of STL – End of Cornelius Pass 42-inch-diameter pipeline	430

Note:
Abbreviation: TVWD - Tualatin Valley Water District.
(1) Minimum HGL elevation based on City of Hillsboro's 2014 revised benchmark elevation of 194.22. JWC transmission main design was originally based on NGVD29 Washington County Benchmark elevation of 194.31.

8.3.2 Partner Demand Assumptions

Five demand scenarios were developed to evaluate the transmission system capacity as follows:

- Existing maximum week.
- 2021 MDD Projection.
- 2042 MDD Projection.
- 85-mgd water treatment plant (WTP) capacity.
- 105-mgd WTP capacity.

The maximum demand week from October 2020 through September 2021 was August 1-7, 2021, as identified from the WTP Daily Summary SCADA data. The existing maximum week demands were developed for each partner connection based on the SCADA data from August 1-7, 2021. The total demand from the maximum demand week was 60 mgd.

For the 2021 MDD and 2042 MDD scenarios (72.5 mgd and 82.8 mgd, respectively), partner demands were developed from Table 5.6 in *Chapter 5 Water Demand Forecast and Supply Strategy*. The partner projections for Beaverton, North Plains, and TVWD match the demands listed in Table 5.6. The TVWD STL connection was assumed to be 4 mgd, since the STL has more limited capacity, with the remaining demand allocated to the TVWD NTL connection.

The 85-mgd WTP capacity scenario represents the current maximum capacity of the WTP of 85 mgd and was included to evaluate the transmission system at maximum production capacity. For this scenario, partner demands were assumed to be the same as the 2042 MDD with the following changes:

- Forest Grove demands were assumed to be 13 mgd to account for an increase in demands from the 2042 MDD scenario.
- Beaverton demands were assumed to be 13.2 mgd to account for an increase in demands from the 2042 MDD scenario.

Similarly, the 105-mgd WTP capacity scenario partner demands were scaled up from the 85-mgd WTP capacity scenario. Note, the demand allocation assumptions of this scenario do not represent partner ownership.

Table 8.3 summarizes the partner connection demand assumptions for the transmission system capacity analysis.

Table 8.3 Partner Connection Demand Assumptions

Partner Connection	Existing Max Week (mgd)	2021 MDD (mgd)	2042 MDD (mgd)	85-mgd WTP (mgd)	105-mgd WTP(1) (mgd)
Hillsboro(2)	28.4	38.2	41.8	41.8	51.5
Beaverton(3)	10.4	12.8	11.5	13.2	16.3
Forest Grove(5)	4.4	5.8	12.6	13.0	16.1
TVWD NTL(4)	12.1	10.5	10.5	10.5	13.0
TVWD STL(3)	4.0	4.0	4.0	4.0	4.9
North Plains(6)	0.6	1.2	2.5	2.5	3.1
Total	**60.0**	**72.5**	**82.8**	**85.0**	**105.0**

Notes:
(1) 105-mgd WTP scenario is scaled up from 85-mgd WTP scenario. This scenario does not represent partner ownership.
(2) See Table 8.4 for demand allocation by Hillsboro connection.
(3) Demand is on the STL.
(4) Demand is on the NTL.
(5) Demand is on the FGHB.
(6) North Plains is not a JWC partner, but as noted in Table 5.6, it is assumed that North Plains' demands can be served out of whichever JWC partner is not utilizing their full allocation. North Plains is served from a connection on the NTL.

The Hillsboro demands were allocated to each Hillsboro connection as outlined in Table 8.4. The demands for the existing maximum week scenario were allocated based on the SCADA data from August 1-7, 2021, consistent with the partner connection demand assumptions for the scenario. The Hillsboro connection demands for the 2021 MDD scenario were scaled up from the existing maximum week scenario to match the overall Hillsboro demand allocation.

To account for the Cornelius demands, which are not included as a separate projection in Table 5.6, the demands for the Cornelius connection was scaled up from the existing maximum week demands.

For the 2042 MDD and 85-mgd WTP capacity scenarios, the Hillsboro connection demands were allocated based on the ratio of flow to each connection to the overall Hillsboro flows shown in Figure 5-11 from the 2019 Hillsboro Water Master Plan. For the 105-mgd WTP Capacity scenario, the Hillsboro connection demands were scaled up from the 85-mgd WTP capacity scenario.

Table 8.4 Hillsboro Connection Demand Assumptions

Hillsboro Connection	Existing Max Week(1) (mgd)	2021 MDD(2) (mgd)	2042 MDD(3) (mgd)	85-mgd WTP(3) (mgd)	105-mgd WTP(4) (mgd)
NTL Valley View	0.3	0.3	0.0	0.0	0.0
NTL Glencoe	2.4	3.2	2.3	2.3	2.8
NTL 25th	2.2	3.0	3.0	3.0	3.8
NTL Dawson	5.4	7.3	10.4	10.4	12.8
NTL Stadium	5.3	7.1	9.0	9.0	11.2
NTL Evergreen	1.2	1.7	0.0	0.0	0.0
NTL Crandall Res	0.6	0.8	0.0	0.0	0.0
STL 1st	2.3	3.1	3.7	3.7	4.6

Hillsboro Connection	Existing Max Week(1) (mgd)	2021 MDD(2) (mgd)	2042 MDD(3) (mgd)	85-mgd WTP(3) (mgd)	105-mgd WTP(4) (mgd)
STL Minter Bridge	1.9	2.6	4.1	4.1	5.1
STL Imlay(5)	2.1	2.8	2.9	2.9	3.6
STL Alexander(5)	1.1	1.5	0.8	0.8	1.0
Blanton (STL)(5)	1.6	2.1	2.8	2.8	3.4
Butternut (STL)(6)	0.2	0.3	0.0	0.0	0.0
Dilley (FGHB)	0.6	0.9	0.8	0.8	1.0
Cornelius (NTL)	1.2	1.6	2.1	2.1	2.6

Notes:

Abbreviations: PRV - pressure-reducing valve; WWSS - Willamette Water Supply System.

(1) Based on 2021 Maximum Week SCADA data.

(2) Scaled up demands by connection based on 2021 Maximum Week SCADA data.

(3) Scaled flows to each connection based on ratio of flows shown in Figure 5-11 from 2019 Hillsboro Water Master Plan.

(4) 105-mgd WTP scenario is scaled up from 85-mgd WTP scenario. This scenario does not represent partner ownership.

(5) The Hillsboro 2019 Water Master Plan includes a capital improvement program project to replace the Alexander and Imlay PRVs with a new valve with two new PRVs. The new station will be balled the 67th PRV.

(6) The existing JWC SCADA data shows demands for the Blanton Hillsboro connection. The Blanton PRV will be served by WWSS when WWSS is online. Hillsboro has a future connection called the 67th PRV on the STL. It was assumed that 2042 MDD, 85-mgd WTP, and 105-mgd WTP demands will be served from the 67th PRV on the STL.

(7) The Butternut Creek neighborhood is currently being served by JWC but will be served by WWSS once the WWSS is online. Demands for Butternut are not included for the 2042 planning year and beyond.

Table 8.5 summarizes the total demands for each demand scenario for the NTL, STL, and FGHB pipelines.

Table 8.5 Demand Scenario Assumptions by Pipeline

Pipeline	Existing Max Week (mgd)	2021 MDD (mgd)	2042 MDD (mgd)	85-mgd WTP (mgd)	105-mgd WTP(1) (mgd)
NTL Total	31.2	36.6	39.8	39.8	49.2
STL Total	23.7	29.2	29.8	31.5	38.9
FGHB Total	5.1	6.7	13.4	13.8	17.1

Note:

(1) 105-mgd WTP scenario is scaled up from 85-mgd WTP scenario. This scenario does not represent partner ownership.

8.3.3 Infrastructure Modeling Assumptions

This section summarizes the assumptions used for infrastructure in the hydraulic model. The level in the Fern Hill reservoirs was assumed to be 11 feet, which is the low-level tank setting in the hydraulic model.

Headloss in the transmission system is a summation of the elevation head, velocity head, and pressure head of water as it moved through the transmission system. Elevation head is the difference in elevation at the start and end of the pipeline. Velocity head accounts for the kinetic energy of the water and increases as the velocity of the water increases. Pressure head includes the friction loss in pipes and the minor losses through pipe fittings.

Pipe material, age, and fittings can affect the pressure head in the transmission system. Hazen-Williams roughness coefficients are used to calculate friction loss in pipes, which contribute to overall headloss through a pipe. Roughness coefficients are assigned to pipes based on the material and age of the pipe. Minor losses due to pipe fittings, such as contractions, valves, and tees, also contribute to overall headloss through a pipe. Friction loss and minor losses are added together to calculate the pressure head through the pipe and will decrease the downstream HGL in the transmission system.

The Hazen-Williams roughness coefficients for the pipeline in the updated and calibrated model are as follows:

- 120 for 24-inch FGHB pipeline.
- 130 for NTL, STL, and N-S Intertie pipeline.

Note that assumed roughness coefficients were not changed in the model.

Minor (head) losses were added to the JWC transmission system based on fittings identified from the as-built drawings of the WTP (including improvements and expansions), the Fern Hill reservoirs, and the pipelines. Minor losses for contractions were calculated using the following equation:

$$Contraction\ Minor\ Loss\ (ft) = 0.5\ (1 - \frac{d_1^2}{d_2^2}) \sqrt{sin\frac{\theta}{2}}$$

where

d_1 = smaller diameter in inches.

d_2 = larger diameter in inches.

θ = angle of contraction.

Minor losses for enlargements were calculated using the following equation:

$$Enlargement\ Minor\ Loss\ (ft) = \ (1 - \frac{d_1^2}{d_2^2})^2$$

Table 8.6 provides the minor loss coefficients assumed for the remaining fixture types. Table 8.7 summarizes the total minor loss coefficient for each pipeline segment.

Table 8.6 Minor Loss Assumptions by Pipe Diameter and Fitting

Fitting	10-inch-diameter (feet)	12-inch to 20-inch-diameter (feet)	24-inch and greater diameter (feet)
Gate valve	0.11	0.1	0.1
Ball valve	0.04	0.04	0.04
Butterfly valve	0.63	0.35	0.3
Air & vacuum(1)	0.63	0.35	0.3
Standard elbow, 90 degrees	0.42	0.39	0.36
Standard elbow, 45 degrees	0.22	0.21	0.19
Miter bends, 30 degrees	0.11	0.1	0.1
Standard tee, thru flow	0.28	0.26	0.24
Standard tee, thru branch	0.84	0.78	0.72

Notes:
(1) Assumed the same minor loss as a butterfly valve.
(2) Source: Cameron Hydraulic Data.

Table 8.7 Minor Loss Assumptions for Pipeline Segments

Location	Minor Loss (feet)
WTP on-site finished water pipeline	20.4
Pipeline from WTP to Fern Hill reservoirs	6.7
NTL, upstream of N-S Intertie	19.2
NTL, downstream of N-S Intertie	12
N-S Intertie	3.1
STL, upstream of N-S Intertie	10
STL, downstream of N-S Intertie	7.8
FGHB	2.1

8.3.4 Results

The results from the hydraulic modeling were compared to the evaluation criteria to determine if JWC's transmission system has sufficient capacity for the projected demands.

No mains in the JWC transmission system reach a velocity of 10 ft/s or greater for any modeled demand scenarios. Maximum velocities in the 85-mgd WTP scenario were as follows:

- Maximum velocity in the NTL/STL/N-S intertie was 6.1 ft/s on the 42-inch-diameter pipeline at the railroad crossing adjacent to SW Dennis Avenue and SW Washington Street and on the 42-inch-diameter pipeline at the railroad crossing between NW Cornell Avenue and NW Glencoe Road.
- Maximum velocity in the 24-inch-diameter FGHB pipeline was 6.6 ft/s.

Pipeline velocity results for the 85-mgd WTP scenario are shown in Figure 8.3, which also shows the demand allocation locations.

The HGL results at the end of the NTL and STL were compared to the HGL requirement as shown in Table 8.8. The HGL at the end of the FGHB pipeline was also included for reference.

Table 8.8 Modeled HGL Results at End of Transmission Mains

Connection	HGL Requirement (ft)	Existing Max Week (ft)	2021 MDD (ft)	2042 MDD (ft)	85-mgd WTP (ft)	105-mgd WTP (ft)
End of NTL(1)	450	485	479	476	475	463
End of STL(2)	430	468	453	453	448	422
End of FGHB	354/390(3)	493	487	449	446	419

Notes:
Abbreviation: ft – feet.
(1) End of NTL is the TVWD connection.
(2) End of STL is the end of the Cornelius Pass 42-inch-diameter pipeline.
(3) HGL for the Hillsboro Upper System is 390 feet. HGL in Forest Grove system at the JWC connection is 354 feet.
(4) Level in Fern Hill reservoirs was assumed to be 11 feet.

As shown in Table 8.8, the HGL requirement at the end of the NTL and STL are met in all modeled demand scenarios except the 105-mgd WTP capacity scenario, where the modeled HGL at the end of the STL is below the requirement. Because the current capacity of the WTP is 85-mgd, no improvements are recommended to address this HGL deficiency. Flows from the JWC WTP to Hillsboro and TVWD connections can also be diverted from the STL to the NTL so there is less flow (and less headloss) through the STL in the future if the actual HGL at the end of the STL approaches the HGL requirement.

Table 8.9 shows the HGL results at all demand connections for each modeled demand scenario for reference.

Table 8.9 Modeled HGL Results at Demand Connections

Connection	Existing Max Week (ft)	2021 MDD (ft)	2042 MDD (ft)	85-mgd WTP (ft)	105-mgd WTP (ft)
NTL Valley View(1)	493	489	n/a	n/a	n/a
NTL Glencoe(1)	488	482	480	479	468
NTL 25th(1)	487	481	478	478	466
NTL Dawson(1)	486	480	477	476	464
NTL Stadium(1)	485	479	476	475	463
NTL Evergreen(1)	486	480	n/a	n/a	n/a
NTL Crandall Res(1)	488	482	n/a	n/a	n/a
STL 1st(1)	487	480	477	476	463
STL Minter Bridge(1)	479	470	468	465	447
STL Imlay(1)	470	457	456	451	427
STL Alexander(1)	470	457	456	451	427
Blanton (STL)(1)	468	453	453	448	422
Butternut (STL)(1,2)	468	453	n/a	n/a	n/a
Dilley(1)/Forest Grove (FGHB)	493	488	450	447	421
Cornelius (NTL)	494	490	489	488	482
Beaverton (STL)	468	453	453	448	422
TVWD NTL	485	479	476	475	463
TVWD STL	467	453	453	447	421
North Plains (NTL)	488	482	479	479	468

Notes:
(1) Hillsboro Connection.
(2) The Butternut Creek neighborhood is currently being served by JWC but will be served by WWSS once the WWSS is online. Results at the Butternut connection are not included for the 2042 planning year and beyond.

The maximum flows for each pipeline segment from the modeled demand scenarios are summarized in Table 8.10.

Table 8.10 Maximum Flows from Each Model Scenario for Pipeline Segment

Location	Existing Max Week (mgd)	2021 MDD (mgd)	2042 MDD (mgd)	85-mgd WTP (mgd)	105-mgd WTP(1) (mgd)
NTL, upstream of N-S Intertie	44	52.4	55.6	56.8	70.1
NTL, downstream of N-S Intertie	29.7	34.6	37.7	37.7	46.6
N-S Intertie	12.8(2)	15.8(2)	15.8(2)	17.0(2)	20.9(2)
STL, upstream of N-S Intertie	10.9	13.3	14.1	14.5	18.0
STL, downstream of N-S Intertie	21.4	26.1	26.1	27.8	34.3
FGHB	5.1	6.7	13.4	13.8	17.1

Notes:
(1) 105-mgd WTP scenario is scaled up from 85-mgd WTP scenario. This scenario does not represent partner ownership.
(2) Flow goes south from NTL to STL.

8.3.5 Finished Water Pump Stations Capacities

The JWC hydraulic model was also used to evaluate the capacity of the finished water pump stations. The total capacity, which is the capacity of the finished water pumps with all pumps online, and the firm capacity, which is the capacity of the finished water pumps with the largest pump out-of-service, were both evaluated using the hydraulic model. Since the existing capacity of the JWC WTP is 85 mgd, the 85-mgd WTP model scenario was used to estimate the pump station capacities. Note, the total design capacity of the nine finished water pumps is 85 mgd as shown in Table 2.10 in *Chapter 2 – Existing System*.

Table 8.11 summarizes the modeled flow out of the Finished Water Pump Station with all pumps online, and with the largest pump out-of-service.

Table 8.11 Finished Water Pump Station Capacities during 85-mgd WTP Model Scenario

Modeled Flow from Finished Water Pumps	Flow (mgd)
Total pump station capacity (all pumps in service)	89.4
Firm pump station capacity (largest pump out-of-service)(1)	75.5

Note:
(1) Assumes Finished Water Pump Station 2 Pump 3 is out-of-service.

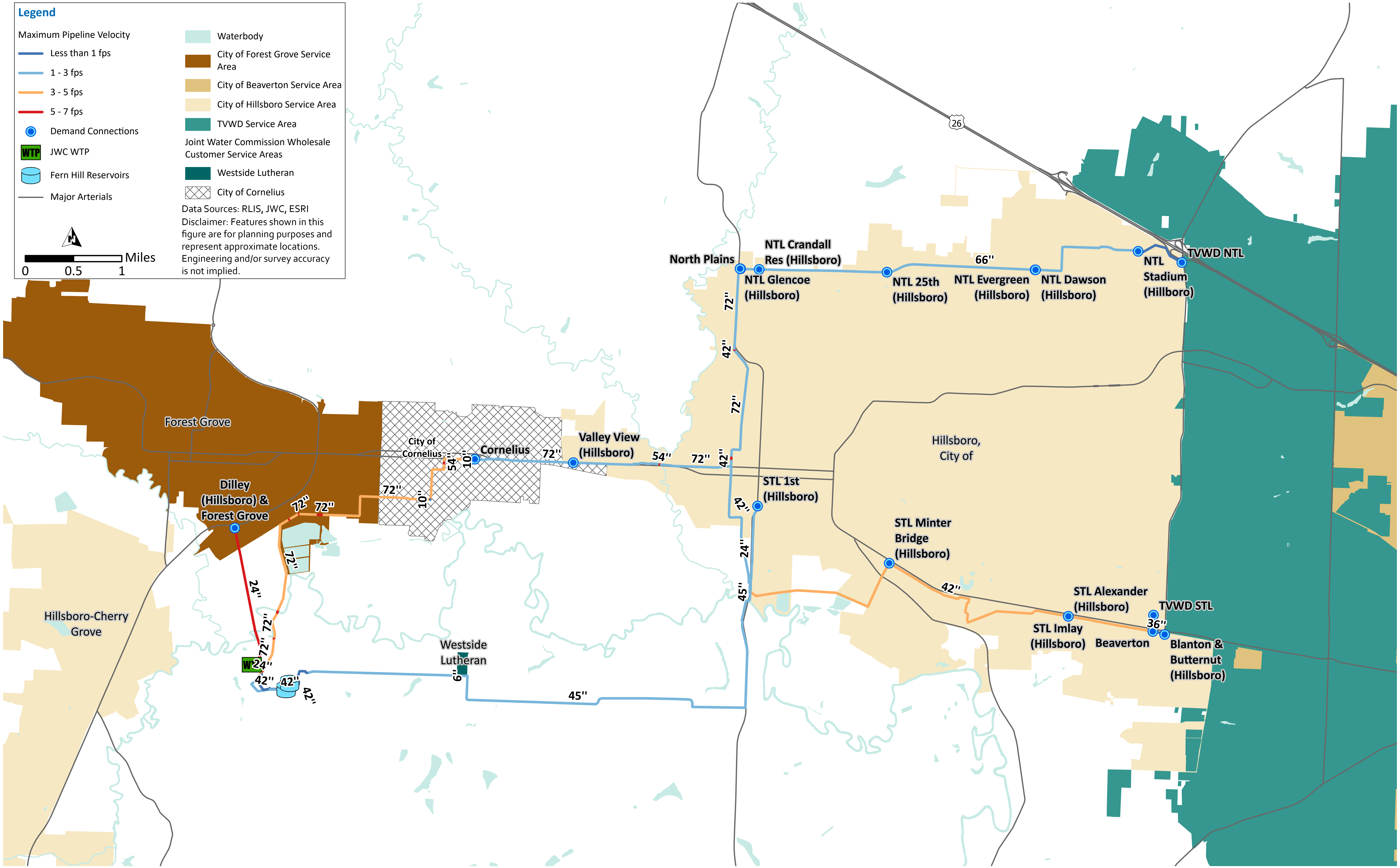
As shown in Table 8.11, the firm capacity of the Finished Water Pump Station is almost 10 mgd below the 85-mgd capacity of the JWC WTP. It is recommended that a fourth pump be installed at Finished Water Pump Station 2 to increase the firm capacity of the pump stations. With the recommended new pump, the modeled firm capacity of the finished water pumps is 89.4 mgd, which is greater than the 85-mgd capacity of the JWC WTP.

8.3.6 Recommendations

The model results indicate that the HGL at the end of the NTL and STL and all modeled pipeline velocities are within JWC's criteria for the 20-year planning horizon. No improvements are recommended to address HGL or velocities in the transmission system.

If the JWC WTP expands capacity to 105 mgd, flows to the Hillsboro and TVWD connections may need to be adjusted to meet the HGL requirement at the end of the STL.

The firm capacity of the finished water pumps is below the currently capacity of the JWC WTP. To increase the firm capacity, a new pump (Finished Water Pump 10) at Finished Water Pump Station 2 is recommended.



Data Sources: RLIS, JWC, ESRI
Disclaimer: Features shown in this figure are for planning purposes and represent approximate locations. Engineering and/or survey accuracy is not implied.

Figure 8.3 Maximum Pipeline Velocity During 85-MGD WTP Capacity Scenario

8.4 Surge Analysis

TM 3: Surge Analysis, included as Appendix H, documents the transient analysis of the JWC finished water pump stations and provides recommendations for surge protection devises. The hydraulic model used to perform the transmission system capacity analysis was also used to perform the surge analysis. The model was used to estimate the likely hydraulic pressure transients during a power outage and evaluate surge mitigation measures.

8.4.1 Summary of Surge Analysis

The surge analysis evaluates the entire JWC system but focuses on the JWC finished water pump stations 1 and 2 and includes Fern Hill reservoirs 1 and 2, the surge vessel near the finished water pump stations, the surge bladder, and air valves.

Surge events are commonly caused by pump station power failure, pump startup, or rapid valve opening or closing events. Water hammer-inducing events were simulated in this study to represent a power failure at the pump stations with the existing surge protection to determine the magnitude of undesirable surge pressures. Then multiple iterations of the power failure event were simulated to test different alternatives until an acceptable surge protection solution was identified.

The results of the surge analysis show that ball valve closures create an upsurge of pressure downstream of finished water pump stations. Extending the closure time would allow reverse flow to continue through the pump station and increase the volume that would reenter the pump station vault. It is recommended that the existing ball valves on the discharge header of finished water pump stations 1 and 2 be replaced with rapid-closing check valves to reduce control discharge pressures at the pump station during a transient event after a pump shutdown.

The model predicts that the surge vessel is not optimally utilized during a surge event. Analyses show the current placement of the surge vessel at Pump Station 2 is not ideal for Pump Station 1 and reduces its effectiveness to protect Pump Station 1 against surges. A new 2,000-gallon surge vessel is recommended near Pump Station 1, located 50 feet or less away from the discharge of Pump Station 1. A 30-inch-diameter connecting pipeline is recommended.

The model also predicts that a rapid closure of the PRV at the Forest Grove connection could cause a sizable surge event within the transmission system. A new 10-inch-diameter surge relief valve is recommended upstream of the Forest Grove pressure reducing valve vault.

Note, during a simulated power shutdown at the finished water pump stations, negative pressure is predicted to occur in the 72-inch-diameter pipeline and 42-inch-diameter pipeline near the Fern Hill reservoirs, but the pressure is not expected to drop below vapor pressure.

8.4.2 Recommendations

Based on the results of the surge analysis, it is recommended that the existing ball valves on the discharge header of finished water pump stations 1 and 2 be replaced with rapid-closing check valves to reduce control discharge pressures at the pump station during a transient event after a pump shutdown. It is also recommended that a new 2,000-gallon surge vessel be installed near Pump Station 1, located 50 feet or less away from the discharge of Pump Station 1. A 30-inch-diameter connecting pipeline is also recommended. Additionally, a new

10-inch-diameter surge relief valve is recommended upstream of the Forest Grove pressure reducing valve vault.

It is recommended that the rapid-closing check valves be installed prior to, or at the same time as, installation of the 2,000-gallon surge vessel.

8.5 Additional Transmission System Improvement Recommendations

This section summarizes recommended improvements from the *2020 Update to JWC's Cathodic Protection Program* (2020 Cathodic Study), *TM 1: Condition Assessment*, and *TM 2: Transmission System Seismic Resiliency Evaluation*. Additional infrastructure improvements identified by the JWC are also included in this section.

8.5.1 2020 Cathodic Study

The 2020 Cathodic Study was completed by Murraysmith and is included as Appendix D. In 2003, JWC engaged Murraysmith and Cascade Corrosion Consulting Services (CCCS) to develop a report entitled *Cathodic Protection Program* (2003 Report) for the existing JWC water transmission mains. The 2003 Report was developed to study approximately 29.5 miles of both CCP and cement mortar lined and coated steel pipeline (CML&C SP).

The 2020 Cathodic Study includes an additional 8.2 miles of pipeline including both CCP and CML&C SP. This piping includes the 42-inch and 36-inch-diameter JWC raw water lines (1993/1974), the 72/54-inch-diameter Fern Hill force main (2005), and the 72-inch-diameter north transmission main (2002). The updated cathodic protection (CP) program examines a total of approximately 37.7 miles of existing JWC water transmission mains.

The 2020 Cathodic Study presents recommendations for repairs to existing test stations (TSs), modifications to the piping appurtenances, installation of additional CP measures, and follow-up testing. Recommendations from the 2003 Report had not been completed when the 2020 Cathodic Study was completed and were still under consideration. The recommended actions from the original 2003 report were retained and listed separately from the recommended actions developed for the newly evaluated pipeline in the 2020 Cathodic Study.

Table 8.12 summarizes the prioritized recommendations from the 2003 Report, and Table 8.13 summarizes the additional recommendations included in the 2020 Cathodic Study. The recommendations are listed in the priority order from the source study with the recommendations from Table 8.12 being higher priority than the recommendations from Table 8.13.

Table 8.12 Prioritized Cathodic Protection Recommendations from the 2003 Report

Item No.	Recommendation	Total Occurrences
1	Install a 32-pound high potential magnesium anode on the blow off assembly piping (STL Phase I, 24" Hillsboro Line, Forest Grove Transmission Main & STL Phase II lines).	24
2	Install an insulating dielectric union of a 32-pound high potential magnesium anode on the air valve piping (STL Phase I, 24" Hillsboro Line, Forest Grove Transmission Main & STL Phase II lines).	37
3	Install an isolation joint on the new fire hydrant lines associated with the STL Phase II line or remove the fire hydrants.	5

Item No.	Recommendation	Total Occurrences
4	Install an isolation joint and TS at the start of the STL Phase I (0+00).	1
5	Install an isolation joint on the 4-inch service west of Golf Creek Rd. (STL Phase 1).	1
6	Install an isolation joint on the 6-inch service off the STL Phase II line (0+20) to Clean Water Services.	1
7	Install flange isolation joint on both the east and west side of meter vault at STL Phase II, 266+20.	2
8	Install an isolation joint and isolation type TS as the 10-inch connection to the Hillsboro (Dilley) PRV station (24" Forest Grove Line).	1
9	Install a 32-pound high potential magnesium anode on the 8-inch abandoned main to the Hillsboro main (24" Forest Grove Line).	1
10	Repair the isolation joint on the by-pass piping inside of the Forest Grove PRV station (Forest Grove Transmission Main).	1
11	Install an isolation joint on the last flange of the Forest Grove Transmission Main before it leaves the north end of the Forest Grove PRV station.	1
12	Install an isolation joint and isolation type TS as the two 42-inch connection to the STL Phase I.	2
13	Install isolation type TS at existing isolation joins – at the following locations: • A. 54" main in WTP and 30" main to Hydropneumatic tank. • B. STL Phase I (435+31+/-) and 24" Hillsboro Line, Forest Grove Transmission Main (0+00). • C. STL Phase I (59+39) and STL Phase II (0+00). • D. STL Phase II (94+00) and 30" connection. • E. STL Phase II (212+55 & 212+60) and 10" & 18" connections. • F. STL Phase II (265+95) and 24" connection.	6
14	Install standard TS along the STL Phase I, 24" Hillsboro Line, Forest Grove Transmission Main, and STL Phase II lines.	44
15	Install small galvanic anode groundbeds at each new standard and isolation type TS (item 13 & 14 above) and install number 2 wire leads for possible future impressed current systems (STL Phase I, 24" Hillsboro Line, Forest Grove Transmission Main, and STL Phase II lines)	54
16	After the installation of the new TSs, perform tests to determine if the mains are electrically continuous and if any additional CP systems are required (STL Phase I, 24" Hillsboro Line, Forest Grove Transmission Main, and STL Phase II lines).	1
17	Install 2-3 impressed current type CP systems on the North-South Intertie, NTL Phase I and NTL Phase II.	2-3

Note:

(1) Source: Table 3-1a Prioritized Recommendations from the 2003 Report as provided in the 2020 CP Program. Table 3-1a also includes location(s) in figures within the report and details (s) for each item as provided in the appendix to the report, and comments on the items.

Table 8.13 Additional Prioritized Cathodic Protection Recommendations (2020)

Item No.	Recommendation	Total Occurrences
18	Install 4 impressed current type CP systems along JWC North Transmission Main. Install joint bonds across the four couplings (two each, either side of railroad tracks). Install TS on both sides with leads going to the main and the casing.	4
19	Install additional TS on both 42” and 36” Raw Water lines (two new per line) for ability to test for continuity.	4
20	After the installation of the new TSs, perform tests to determine if the mains are electrically continuous and if any additional CP systems are required (Raw Water Lines).	2
21	After TSs 23-26 are located or reinstalled on the STL Phase II Line, perform stray current tests while NW Natural interrupts their rectifiers at SE 44th Ave & TV Hwy and TV Hwy & 198th Ave. If results show no stray current, CP will need to be installed for possible major corrosion around STL21 & STL22.	4
22	Perform test to determine if 72” WTP to Fern Hill are electrically continuous and if any additional CP system are required. Install test leads between the couplings and butterfly valve (both locations).	1

Note:

(1) Source: Table 3-1b Additional Prioritized Recommendations (2020) provided in the 2020 CP Program. Table 3-1b also includes location(s) in figures within the report and details (s) for each item as provided in the appendix to the report, and comments on the items.

8.5.2 TM 1: Condition Assessment

TM1: Condition Assessment was developed as part of this Master Plan and is included as Appendix E. This TM reviews the results and recommendations of previously conducted condition assessments, leak and event history reports, and master plans to understand the transmission lines’ general condition and susceptibilities. The TM outlines potential assessment and inspection methods that JWC may employ in the future and makes specific recommendations for the STL, NTL, and the FGHB transmission main, which are summarized in this section.

8.5.2.1 STL and FGHB Recommendations

Recommendations for the STL and FGHB include identifying new high-risk areas of the STL by conducting a second close interval survey (CIS) assessment as outlined in the 2008 Condition Assessment and identifying high-risk areas of the FGHB by conducting a first CIS assessment. The CIS assessment should be conducted in the short-term, to identify high-risk areas of continuing corrosion and provide an overall update on system conditions.

The high-risk areas identified using the CIS evaluation should be evaluated using remote field eddy current/transformer coupling (RFEC/TC). The information gathered using RFEC/TC can be utilized to determine the pipeline conditions that may lead to leaks and ruptures.

In conjunction with the CIS evaluation and RFEC testing to identify and assess high-risk areas, installations of piezoelectric hydrophones for ultrasonic testing in both newly and previously identified high-risk areas is recommended for on-going leak monitoring. Additional assessment of any new high-risk areas, not previously identified as experiencing joint mortar degradation in prior studies, should be inspected using non-destructive methods to determine the extent of the mortar coating corrosion, including:

- Concrete and mortar sounding by tapping the surface with a hammer and listening for audible discontinuities.
- Concrete and mortar sampling to determine the physical and chemical properties, using a scraper to gently remove a few ounces of material.
- Ultrasonic testing using an external transducer to determine wall thickness.

Ultrasonic testing is recommended to estimate the wall thickness of the pipeline and identify areas experiencing pitting corrosion. Because ultrasonic testing requires excavating the pipeline and removing its coating, this technology should only be used for high-risk locations. Future monitoring locations can be determined using leak history and other contributing factors.

8.5.2.2 NTL and N-S Intertie Recommendations

The NTL and STL require separate monitoring programs due to the difference in age and material of these pipelines. The NTL and N-S Intertie all made from MLCS, so the following recommendations apply to both pipelines. The 2009 Master Plan recommended ongoing monitoring of the NTL for the next 10 years, including soil resistivity testing and ultrasonic testing for leak detection and wall thickness determination. Soil resistivity testing and analysis is recommended to determine soil characteristics along the length of pipeline. Results can be used to project concrete-coating deterioration rates, estimate pipeline life, and identify potential high-risk areas of corrosion.

As with the STL, installation of piezoelectric hydrophones for ultrasonic testing in high-risk areas for leak monitoring is recommended. Ultrasonic testing is recommended to estimate the wall thickness of the pipeline and identify areas experiencing pitting corrosion, thereby producing a more detailed assessment of the condition of the pipeline. Because ultrasonic testing requires excavating the pipeline and removing its coating, this technology should only be used for high-risk locations. Future monitoring locations can be determined using leak history and other contributing factors.

8.5.3 TM 2: Transmission System Seismic Resilience Evaluation

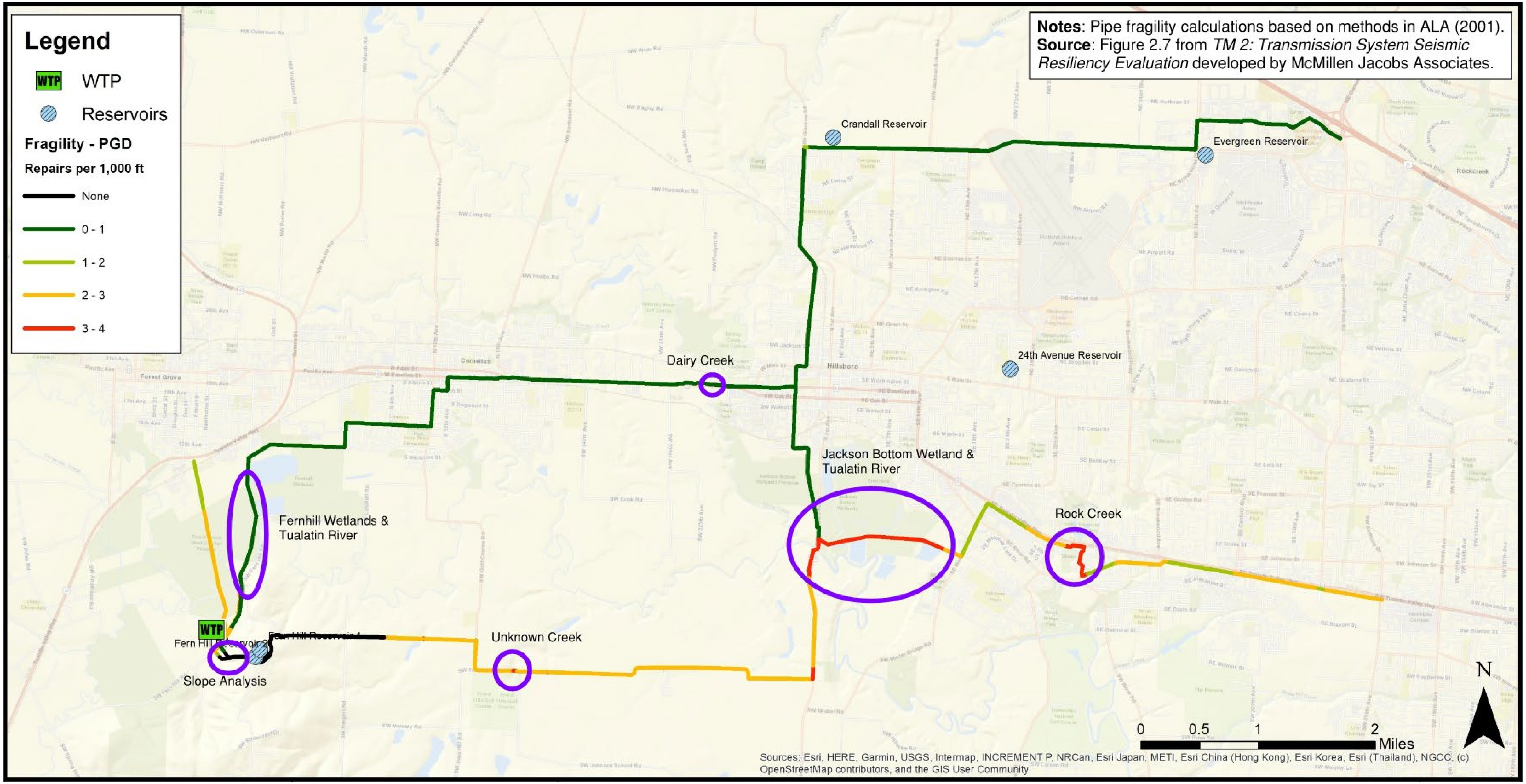
McMillen Jacobs Associates (MJA) was retained to provide evaluation, analysis, and recommendations regarding the seismic resiliency portion of this Master Plan. *TM 2: Transmission System Seismic Resiliency Evaluation* summarizes the work and is included as Appendix F.

Based on the seismic hazards assessment and the pipeline fragility evaluation, MJA concludes that some portions of the JWC transmission pipelines will be highly vulnerable to hazards associated with a magnitude 9.0 Cascadia Subduction Zone (CSZ) earthquake and the Maximum Considered Earthquake (MCE). The MCE considers the probabilities and potential ground motion intensities of earthquakes in the project area with a recurrence period of 2,475 years. The anticipated damagers (in terms of pipeline leaks and breaks) will mainly occur in areas with the large seismic permanent ground deformation (PGD), such as in high liquefaction potential areas, medium to large lateral spreading zones, and in or near high seismic landslide hazard zones. Additionally, the strong shaking from an MCE type of seismic event will have a more significant impact on pipeline integrity than a CSZ event. Note, the MCE is typically used as the design earthquake for critical structures and facilities and recently has been considered for the designs of some critical pipelines and transmission mains such as Willamette Water Supply Project and Bull Run Filtration Facility Pipelines.

To improve the seismic resiliency of JWC's transmission pipelines, MJA recommends the following strategies:

- Evaluate site-specific PGD and pipeline seismic performance analyses at high seismic PGD hazard areas, such as steep slope areas along Fern Hill Road, Tualatin River Crossing and associated wetland areas, and other major creek crossings. The seismic analysis for the pipeline should consider the non-linear behaviors of the soil/pipe interaction and pipe materials, pipe's internal pressure, joint effect, and should be a strain-based evaluation. Typically, these evaluations are accomplished using Finite Element Analysis numerical modeling approaches.
- Evaluate the seismic performance of the pipelines under strong ground shaking (i.e., MCE) to assess the potential impact to the pipelines and joints. For the lap-welded NTL and Intertie pipelines, the induced stress/strain at the pipes and joints are not expected to be significant; however, for the STL with rubber gasket joints, the effect may be significant.
- Based on the pipeline seismic analyses results, replace the crossings or critical sections of the pipelines which are deemed to have high risk of failure with seismic resilient pipelines, such as welded steel pipelines or Earthquake Resilient Ductile Iron Pipes (ERDIP). Localized ground improvements, pile supports, or pipeline realignment are also options to reduce the impacts of PGD.
- Seismically improve the existing Intake and replace raw water CCP pipelines with seismic resilient pipelines, such as welded steel pipelines or ERDIP. Alternatively, new seismically resilient intake and raw water pipelines can be constructed at a different location.
- Within the scheduled resiliency improvement period suggested by the Oregon Resiliency Plan (ORP) (in the next 40 to 45 years) systematically replace the STL CCP pipelines with seismic resilient pipeline, such as welded steel pipelines or ERDIP. Alternatively, if deemed feasible, add joint restraints to the STL CCP pipelines to resist the strong ground shaking effect and differential liquefaction settlement effect.
- Improve seismic resiliency of pressure reducing valve vaults and other appurtenances.

Figure 8.4 shows the pipeline fragility – PGD map with the locations of river/creek crossing and other high hazard areas identified (source for Figure 8.4 is Figure 2.7 from TM 2 developed by McMillen Jacobs Associates).



Notes: Pipe fragility calculations based on methods in ALA (2001).
Source: Figure 2.7 from *TM 2: Transmission System Seismic Resiliency Evaluation* developed by McMillen Jacobs Associates.

Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

Figure 8.4 Pipe Fragility - PGD Map with Locations of River/Creek Crossing and Other High Hazard Areas Identified

8.5.4 Other Transmission System Improvements

Additional transmission system improvements have been identified by JWC and are summarized below:

- Bi-directional flow meter on the N-S Intertie.
- Seismic valves on the following pipelines:
  - N-S Intertie.
  - 54-inch-diameter pipeline from WTP to Fern Hill Reservoir 2, located near the reservoir.
  - 42-inch-diameter pipeline from Fern Hill Reservoir 2 to the transmission system, located near the reservoir.
  - 30-inch-diameter overflow/drain pipeline out of Fern Hill Reservoir 2.
- Clear transmission main corridor easement of overgrowth brush.
- Beaverton and TVWD billing meters: Find locations on the NTL and STL to install new, JWC-owned billing meters that are separate from the partner meters:
  - Beaverton recommends a new JWC meter at the interface of the Beaverton 36-inch-diameter CCP pipeline.
  - TVWD NTL: A billing meter is included in facility upgrades that are planned with the Cornelius Pass Road Pipeline (PLW 2.0), which is scheduled for 2036. Note, a meter to TVWD after Hillsboro connections could be included in the Stadium improvements project.
  - TVWD STL: Location for meter needs to be determined.

In the 2009 Master Plan, purchasing spare steel pipe for the transmission line was recommended. JWC currently has the following spare steel pipe sections and accessories:

- Three sections of 72-inch-diameter pipe.
- Four sections of 42-inch-diameter pipe.
- One section of 54-inch-diameter pipe.
- Butt strap joints.

It is recommended that JWC procure the following additional spare parts: 45-inch-diameter CCP for the STL and butterfly valves.

8.5.4.1 Valve Exercising and Bypass Valves

Valve exercising is also recommended to ensure the valves on the NTL, STL, N-S Intertie, and FGHB can be closed. For the NTL, N-S Intertie, and STL loop, backpressure should be available so that pressure on the loop can be maintained while valves are exercised. *The American Water Works Association (AWWA) M44 Distribution Valves: Selection, Installation, Field Testing, and Maintenance, Third Edition* recommends that a condition assessment and operation of each critical valve and all valves 16-inches and larger should be completed on a regularly scheduled (annual, if possible, for partial closure) basis. AWWA M44 also recommends that all gate valves should be physically cycled from full open to close and back open at least once every five years.

If exercising valves at locations without backpressure, such as the FGHB, the pipeline to North Plains, or the east end of the NTL until WWSS PLW 2.0 is online, small bypass valves and associated bypass piping can facilitate moving water around the large diameter valves. JWC has approximately 16 large-diameter valves on the un-looped NTL, STL, and FGHB where small bypass valves and associated bypass piping are recommended. Note, bypass valves are not necessary on the un-looped NTL, STL, and FGHB pipeline if JWC is willing to use backpressure from WWSS (after 2026) and partners' water systems.

Coordination with downstream connections to select low-demand periods where the system could operate on the bypass valves enabling the large diameter valve to be fully actuated should be considered.

8.5.4.2 Emergency Interties

Additional supply sources and emergency interties for the partner agencies, North Plains, and Cornelius are also reviewed to determine if any new emergency interties are needed. Supply sources (outside of JWC) and emergency interties for the agencies are as follows:

- Hillsboro has nine small interconnections with TVWD and one interconnection with Cornelius for emergency supply of water. Hillsboro will also have the WWSS as an additional source, which is planned to be online in 2026.
- Beaverton operates two ASR wells with three more currently under development. Beaverton also maintains interties with the City of Tigard, the Portland Water Bureau (PWB), and TVWD, which are currently used only for emergencies. Beaverton will also have the WWSS as an additional source, which is planned to be online in 2026.
- TVWD currently purchases water from PWB and has three existing groundwater supply wells and one ASR well. TVWD maintains a network of emergency interties with PWB, Hillsboro, Beaverton, Tigard, and other municipalities. TVWD will also have the WWSS as an additional source, which is planned to be online in 2026. It is expected that TVWD's main connection with PWB will become an emergency interconnection once the WWSS comes online.
- Forest Grove has its own water treatment plant. Forest Grove also has an emergency intertie with Cornelius and an emergency intertie point on the NTL with plans to construct a PRV connection to the emergency intertie on the NTL in 2022.
- North Plains has one well that serves as an emergency backup supply source.
- Cornelius has a small intertie with Hillsboro and an ASR well.

As additional emergency interties are the responsibilities of the individual water systems, the partners and other wholesale customers will need to evaluate if the existing emergency interties are sufficient. JWC will not be responsible for building any additional emergency interties.

8.6 Summary of Transmission System Recommendations

Table 8.14 summarizes the recommended transmission system improvements presented in the chapter to be included in the Capital Improvement Program.

Table 8.14 Summary of Transmission System Recommendations

Project ID	Category	Description
T-26	Pump Station Capacity	Install new pump (Finished Water Pump 10) at Finished Water Pump Station 2 to increase the firm capacity of the finished water pumps (capacity with the largest pump out-of-service).
D-1	Cathodic Protection	Complete all CP recommendations from the 2020 Cathodic Study, including the prioritized recommendations from the 2003 Report and the additional prioritized recommendations from the 2020 Report.
D-2	Pipeline Infrastructure	Install bi-directional flow meter on N-S intertie.
D-3	Pipeline Infrastructure	Install four total seismic valves at the following locations: • N-S Intertie. • 54-inch-diameter pipeline from WTP to Fen Hill Reservoir 2. • 42-inch-diameter pipeline from Fern Hill Reservoir to transmission system. • 30-inch-diameter overflow/drain pipeline out of Fern Hill Reservoir.
D-4	Surge Analysis	Replace existing ball valves on the discharge header of the finished water pump stations 1 & 2 with rapid-closing check valves to reduce control discharge pressures at the pump stations during a transient event after a pump shutdown.
D-5	Surge Analysis	Install new 2,000-gallon surge vessel near finished water pump station 1 located 50 feet or less from the discharge of the pump station. Install 30-inch-diameter connecting pipeline. This project should be completed at the same time, or after, installation of the rapid-closing check valves.
D-6	Surge Analysis	Install new 10-inch-diameter surge relief valve upstream of the Forest Grove pressure reducing valve vault to reduce surge event in the transmission system.
D-7	Pipeline Condition	Transmission Line Condition Assessment Program: • Conduct CIS assessment on the STL and FGHB to identify high-risk areas. Conduct soil resistivity testing and analysis on the NTL and N-S Intertie to determine the soil characteristics along the length of the pipe. • Conduct RFEC testing to determine pipe conditions that may lead to leaks and ruptures on the STL on high-risk areas identified in the 2003 Study & the CIS assessment and on the FGHB on high-risk areas identified in the CIS assessment. • Install piezoelectric hydrophones for ultrasonic testing in the STL, NTL, N-S Intertie, and FGHB in high-risk areas for ongoing leak monitoring. Ultrasonic testing is recommended to estimate the wall thickness of the pipe of both CCP and MLCS pipe and to identify areas experiencing pitting corrosion in MLCS pipe. If installing piezoelectric hydrophones on the STL and FGHB, also conduct concrete and mortar sounding and sampling concurrently.

Project ID	Category	Description
D-8	Pipeline Infrastructure	Program to replace or improve transmission lines (within 20-year CIP)
D-9	Pipeline Infrastructure	Install Beaverton and TVWD billing meters (three total).
D-10	Seismic Resiliency	Perform site-specific geotechnical and pipeline analyses at high hazard areas.
D-11	Seismic Resiliency	Replace or improve pipeline in critical high hazard areas (rivers/creek crossings) based on site-specific analyses.

Chapter 9

CAPITAL IMPROVEMENT PROGRAM

9.1 Introduction

This chapter combines the various projects recommended in the previous chapters of this Water System Master Plan (Plan) and previous studies and tasks into a comprehensive capital improvement program (CIP). The CIP consists of costs estimates and prioritization for each recommended improvement project. The CIP serves as a guideline for the future planning, budgeting, and financial evaluation.

This Plan encompasses a 20-year planning horizon and includes a 20-year CIP through fiscal year (FY) 2042-43. The Plan also includes a 40-year resilience CIP, which includes projects beyond the 20-year planning horizon in Phase 4: FY2043-44 through FY 2062-63.

9.1.1 Capital Project Categories

JWC classifies capital projects into the following categories:

- Treatment Facilities.
- Supervisory Control and Data Acquisition (SCADA).
- Land.
- Transmission Lines.
- Water Rights.
- Machinery & Equipment.
- Reservoirs.

Recommended improvement projects from this Plan were classified based on the Joint Water Commission's (JWC's) project categories.

9.1.2 Project Drivers

The project drivers, or benefits, for each project were also identified to further define the recommended improvement projects. The projects were assigned one or more of the following project drivers:

- Safety.
- Age/Condition.
- Operation & Maintenance (O&M).
- Capacity.
- Water quality.
- Resilience.

9.2 Development of the Capital Improvement Program

This section summarizes the recommended improvement projects and the source of the project. The section is organized by capital project category. Note, no recommended improvement projects included in the CIP were classified as machinery & equipment or reservoirs, projects.

9.2.1 Treatment Facilities

Recommended improvement projects at the JWC Water Treatment Plant (WTP) are summarized in Table 9.1. Treatment facility projects were taken from several sources, including the following:

- *Technical Memorandum No. 1 Capital Improvement Program Update* (2015 CIP Update).
- *Water Treatment Facility Plan* (2018 Facility Plan):
  - Projects primarily consist of capital improvement maintenance projects (CIMP) for 2020 through 2026 and phase II seismic replacement projects.
- *Finished Water Pump Station 1, Pump and Motor Replacement Preliminary Design Report*.
- The following chapters and technical memoranda (TM) of this Plan:
  - Chapter 2: Existing System.
  - Chapter 6: Intake Evaluation.
  - Chapter 7: WTP Evaluation.
  - Chapter 8: Transmission System Evaluation.
  - TM 3: Future Water Quality Blending Evaluation.
  - TM 5: Electrical System Condition Assessment.
- JWC's FY 2023 10-year CIP.
- Discussions with JWC staff.

Table 9.1 Treatment Facilities CIP Projects

ID	Name	Source	Driver
E-01	Remove and Install New Cables on Old SWGR	TM 5	Age/Condition
E-02	Electrical Studies	TM 5	Age/Condition, Resilience, Capacity
E-03	Replace MCC MCS and Panel LGG	TM 5	Age/Condition
E-04	Replacement of Circa 1998 Electrical Equipment	TM 5	Age/Condition
E-05	Replace MCC MCS and MCO	TM 5	Age/Condition
E-06	JWC Electrical Improvements Identified in Study (Includes FWPS2 Transformer)	Discussions with JWC Staff	Age/Condition
T-01	Add Additional PRS Station	JWC's FY2023 10-yr CIP	Water Quality
T-02	Land Purchase for Future Intake	JWC's FY2023 10-yr CIP	Resilience
T-03	Hazard Mitigation Study	Chapter 3	Resilience
T-04	New Intake or SHPP Upgrade	Chapter 6	Capacity, Resilience
T-05	Initiate Study with USBOR for SHPP	Discussion with JWC Staff	Capacity, Resilience
T-06	Hazard Mitigation Study Implementation	JWC's FY2023 10-yr CIP	Resilience
T-07	Replace Rapid Mix Facility	2018 Facility Plan	Resilience
T-08	Replace Floc/Sed Basins A-C	2018 Facility Plan	Resilience
T-09	CIMP: Add Hatches to Floc/Sed Basins	2018 Facility Plan	O&M
T-10	Seismic: Structurally Strengthen Basins D-G Baffles	2018 Facility Plan	Resilience

ID	Name	Source	Driver
T-11	Replace Filters 1-14	2018 Facility Plan	Resilience
T-12	Replace Existing Clearwell	2018 Facility Plan	Capacity, Resilience
T-13	FWPS 2 Seismic Improvements	2018 Facility Plan	Capacity, Resilience
T-14	CIMP: Replace FW Pump 7	2015 CIP Update	Age/Condition
T-15	CIMP: Replace FW Pump 8 and BW Pump 3	2015 CIP Update	Age/Condition
T-16	CIMP: Replace FW Pump 9 and BW Pump 4	2015 CIP Update	Age/Condition
T-17	Construct Second Washwater Surge Basin	2018 Facility Plan	Capacity, Resilience
T-18	Replace Chlorine Gas System	Chapter 7	Safety, Capacity, O&M
T-19	Install OSHG Equipment into Hypo Facility	Chapter 7	Safety, Capacity, O&M
T-20	Construct New Chemical Building	2018 Facility Plan	Resilience, Capacity
T-21	Replace Operations and Maintenance Building	2018 Facility Plan	Age/Condition, Resilience, Safety
T-22	Hydropneumatic Surge Tank Liquefaction Mitigation	2018 Facility Plan	Resilience
T-23	Update JWC CIP Master Plan Every 5 Years	2018 Facility Plan	All
T-24	15,000-Gallons Diesel Fuel Tank(1)	Discussions with JWC Staff	Capacity, Resilience
T-25	New Alum Tank	Discussions with JWC Staff	Water Quality, Capacity
T-26	New Finished Water Pump 10 with VFD	JWC's FY2023 10-yr CIP, Chapter 8	Capacity
T-27	Sedimentation Basins A/B/C Concrete	JWC's FY2023 10-yr CIP	Age/Condition
T-28	PAC Injection System Expansion	JWC's FY2023 10-yr CIP	Water Quality, Capacity

Note:

(1) Based on the wildfire incident JWC had in 2020, 15,000 gallons would provide a two-day fuel of approximately 55 MGD. The additional 15,000 gallons would give JWC more fuel to provide water demand during high demands and emergencies. The tanks would have the same polishing system as the existing system.

Abbreviations: BW – backwash; Floc/Sed – flocculation and sedimentation; FW – finished water; FWPS – finished water pump station; MCC – motor control center; OSHG – on-site hypochlorite generation; PAC – powdered activated carbon; SHPP – Spring Hill Pumping Plant; SWGR – switchgear; USBOR – Bureau of Reclamation; VFD – variable frequency drive.

9.2.2 SCADA

One SCADA program is included in the CIP and summarized in Table 9.2. This project is a program to complete all projects identified in the *Supervisory Control and Data Acquisition (SCADA) Master Plan*.

Table 9.2 SCADA CIP Projects

ID	Name	Source	Driver
S-01	SCADA Program	SCADA Master Plan	Resilience, Age/Condition, O&M

9.2.3 Land

One project categorized as a land project is included in the CIP and summarized in Table 9.3. This project is referenced in *Chapter 6 – Intake Evaluation* and is identified in JWC's FY 2023 10-year CIP.

Table 9.3 Land CIP Projects

ID	Name	Source	Driver
T-03	Land Purchase for Future Intake	JWC FY 2023 10-yr CIP	Resilience

9.2.4 Transmission Lines

Recommended improvement projects related to JWC's transmission lines are summarized in Table 9.4 and were taken from several sources, including the following:

- *2020 Update to JWC's Cathodic Protection Program* (2020 Cathodic Study).
- The following chapters and TM of this Plan:
  - Chapter 8: Transmission System Evaluation.
  - TM 1: Condition Assessment Framework.
  - TM 2: Transmission System Seismic Resilience Evaluation.
  - TM 4: Surge Analysis.
- Discussions with JWC Staff.
- JWC's 10-year CIP for FY 2023.

Table 9.4 Transmission Lines CIP Projects

ID	Name	Source	Driver
D-01	Cathodic Protection Program	2020 Cathodic Study	Age/Condition
D-02	Bi-Directional Flowmeter	Discussion with JWC Staff	O&M
D-03	Seismic Valves on N-S Intertie and Fern Hill Force Mains	Discussion with JWC Staff	Resilience
D-04	Rapid Closing Check Valves at FWPSs	TM 4	O&M
D-05	2,000-Gallon Surge Vessel	TM 4	O&M
D-06	Install Surge Relief Valve Upstream of Forest Grove PRV	TM 4	O&M
D-07	Transmission Line Condition Assessment Program	TM 1	Age/Condition
D-08	Replace/Improve Transmission Lines in Critical High-Hazard Areas (20-yr CIP)	TM 2	O&M
D-09	Beaverton and TVWD Billing Meters	Discussion with JWC Staff	O&M
D-10	Geotechnical and Pipeline Analyses	TM 2	Resilience
D-11	Replace/Improve Transmission Lines in Critical High-Hazard Areas Past (20-yr CIP)	TM 2	O&M

Note:
Abbreviations: N-S Intertie - north-south intertie; PRV - pressure-reducing valve; TVWD - Tualatin Valley Water District.

9.2.5 Water Rights

Two projects categorized as water rights projects are included in the CIP and summarized in Table 9.5. These projects were discussed with JWC staff or included in JWC's FY2023 10-Year CIP

Table 9.5 Water Rights CIP Projects

ID	Name	Source	Driver
R-01	Water Rights: S-55219	Discussion with JWC Staff & JWC's FY2023 10-Year CIP	Capacity
R-02	Water Rights: S-88506	Discussion with JWC Staff & JWC's FY2023 10-Year CIP	Capacity

9.2.6 Project Details

Individual project sheets were created for each CIP project and include project timing, description, estimated cost, risk calculation, spending schedule, and ownership allocation to aid in future implementation. The project sheets are included in Appendix M.

9.3 Cost Estimating

Recommended CIP projects identified in previous studies and plans included cost estimates. These costs estimates have been escalated from the original cost estimate year to 2022 dollars using the July 2022 Engineering News-Record's U.S. 20-City Construction Cost Index of 13168.

Cost estimates were developed for recommended projects identified in this Plan. These cost estimates are planning level costs approximately equivalent to Association for Advancement of Cost Engineering Class 5 estimates. Cost estimates of this type are classified as order-of-magnitude cost estimates, which assume a 0 to 2 percent level of project definition to reflect the significant number of unknowns in project scope and conditions. Correspondingly, Class 5 cost estimates have a wide accuracy range to reflect these uncertainties at the master planning stage; actual costs may vary from these by minus 50 percent to plus 100 percent:

- **Low End Accuracy Range**: -20 to -50 percent (i.e., the low end of the accuracy range for a $1 million ($M) cost estimate is $0.5 to $0.8M).
- **High End Accuracy Range**: +30 to +100 percent i.e., the high end of the accuracy range for a $1M cost estimate is $1.5 to $2.0M).

All CIP costs are presented in 2022 dollars. The individual project sheets include the original cost estimate, source of the cost estimate, the year and 20-City Construction Cost Index for the original cost estimate, and notes on the cost estimate as needed.

9.4 Project Prioritization and Phasing

Recommended CIP projects were reviewed and prioritized according to risk scores and categorizes into CIP phases. This section summarizes the project prioritization and phasing process.

9.4.1 Project Prioritization

A methodology to prioritize the recommended CIP projects was developed by considering the consequence of failure and likelihood of failure of the infrastructure or asset involved.

Consequence of failure, or criticality, describes what effect a failure of the infrastructure or asset would have on the JWC system. Criticality considers the ability and cost to mitigate, the cost of repairs, effect of the failure on water system functionality, the effect of the failure on the built environment, and the effect of the failure on the natural environment. Each project was assigned a criticality score from 1 to 5 as follows:

- 1 (negligible): limited to no consequence of failure.
- 2 (low): facility has redundancy, minor impacts to system.
- 3 (moderate): limits potable water production capacity.
- 4 (high): inability to produce potable water.
- 5 (severe): life safety.

Likelihood of failure, or vulnerability, describes how likely an asset is to fail. Vulnerability considers the age of the asset or its remaining useful life, maintenance records, estimated condition, and the estimated seismic damage. Each project was assigned a vulnerability score from 1 to 5 as follows:

- 1 (negligible): failure of facility is not likely to occur.
- 2 (low): low likelihood of failure. For seismic, earthquake likelihood for 475-year earthquake (10 percent chance in 50 years).
- 3 (moderate): likelihood of event that could damage facility is higher than earthquake (e.g., potential for leak in transmission line).
- 4 (high): facility is at, or near end of its useful life.
- 5 (severe): facility is beyond the end of its useful life.

The project criticality and vulnerability scores were multiplied together to calculate a “risk score” from 1 to 25. The risk score was then used to help prioritize the recommended CIP projects.

9.4.2 Project Phasing

After each recommended CIP project was assigned a risk score, the projects were prioritized by JWC staff in a workshop setting. Then the projects were assigned to CIP implementation phases according to their priority. Within the planning horizon for the 20-year CIP, project timing was subdivided into the following phases:

- Phase 1: FY 2023-24 through FY 2027-28.
- Phase 2: FY 2028-29 through FY 2032-33.
- Phase 3: FY 2033-34 through FY 2042-43.

Resilience projects to be implemented beyond the 20-year planning horizon were assigned to Phase 4: FY2043-44 through FY 2062-63. The individual project sheets include both the calculated risk score and timing based on the estimated project phase.

9.5 Capital Improvement Program

The recommended CIP projects are summarized in Table 9.6. The table presents detailed costs for each year of Phase 1 and Phase 2 (FY 2023-24 through FY 2032-33) and total costs for Phase 3 and Phase 4. The table also summarizes the partner allocation for each project.

Table 9.6 CIP Summary

No.	Project Name	Category	Priority	Total CIP Cost Estimate	FY 23/24	FY 24/25	FY 25/26	FY 26/27	FY 27/28	FY 28/29	FY 29/30	FY 30/31	FY 31/32	FY 32/33	Phase 1 (FY2024-2028)	Phase 2 (FY2029-2033)	Phase 3 (FY2034-2043)	Phase 4 (FY2044-2063)	Hillsboro Ownership	Forest Grove Ownership	Beaverton Ownership	TVWD Ownership
CIP Total				$275,419,000	$3,609,000	$6,778,000	$6,462,000	$4,201,000	$2,441,000	$4,095,000	$3,406,000	$3,496,000	$3,822,000	$3,322,000	$23,490,000	$18,141,000	$92,805,000	$140,983,000	$134,864,000	$32,354,000	$60,725,000	$47,476,000
Annual Cost															$4,698,000	$3,628,000	$9,281,000	$7,049,000	$6,743,000	$1,618,000	$3,036,000	$2,374,000
D-01	Cathodic Protection Program	Transmission Lines	High	$2,133,000	$ -	$237,000	$237,000	$237,000	$237,000	$237,000	$237,000	$237,000	$237,000	$237,000	$948,000	$1,185,000	$ -	$ -	49%	12%	22%	17%
D-02	Bi-Directional Flowmeter	Transmission Lines	Medium	$300,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$300,000	$ -	49%	0%	3%	49%
D-03	Seismic Valves on N-S Intertie and Fern Hill Force Mains	Transmission Lines	Low	$1,170,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$585,000	$585,000	$ -	$1,170,000	$ -	$ -	49%	12%	22%	17%
D-04	Rapid Closing Check Valves at Finished Water Pump Stations	Treatment Facilities	High	$600,000	$ -	$ -	$ -	$600,000	$ -	$ -	$ -	$ -	$ -	$ -	$600,000	$ -	$ -	$ -	49%	12%	22%	17%
D-05	2,000-Gallon Surge Vessel	Treatment Facilities	Medium	$400,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$400,000	$ -	49%	12%	22%	17%
D-06	Install Surge Relief Valve Upstream of Forest Grove PRV	Transmission Lines	Medium	$300,000	$ -	$ -	$ -	$300,000	$ -	$ -	$ -	$ -	$ -	$ -	$300,000	$ -	$ -	$ -	50%	50%	0%	0%
D-07	Transmission Line Condition Assessment Program	Transmission Lines	High	$2,217,000	$ -	$ -	$ -	$739,000	$739,000	$739,000	$ -	$ -	$ -	$ -	$1,478,000	$739,000	$ -	$ -	49%	12%	22%	17%
D-08	Replace/Improve Transmission Lines in Critical High-Hazard Areas (20-yr CIP)	Transmission Lines	High	$30,000,000	$ -	$ -	$ -	$ -	$ -	$2,000,000	$2,000,000	$2,000,000	$2,000,000	$2,000,000	$ -	$10,000,000	$20,000,000	$ -	49%	12%	22%	17%
D-09	Beaverton and TVWD Billing Meters	Transmission Lines	Low	$900,000	$ -	$ -	$ -	$900,000	$ -	$ -	$ -	$ -	$ -	$ -	$900,000	$ -	$ -	$ -	0%	0%	33%	67%
D-10	Geotechnical and Pipeline Analyses	Transmission Lines	High	$750,000	$375,000	$375,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$750,000	$ -	$ -	$ -	49%	12%	22%	17%
D-11	Replace/Improve Transmission Lines in Critical High-Hazard Areas (Past 20-yr CIP)	Transmission Lines	Medium	$30,000,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$30,000,000	49%	12%	22%	17%
E-01	Remove and Install New Cables on Old SWGR	Treatment Facilities	Very High	$446,000	$ -	$223,000	$223,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$446,000	$ -	$ -	$ -	49%	12%	22%	17%
E-02	Electrical Studies	Treatment Facilities	Medium	$50,000	$ -	$ -	$ -	$ -	$ -	$ -	$50,000	$ -	$ -	$ -	$ -	$50,000	$ -	$ -	49%	12%	22%	17%
E-03	Replace MCC MCS and Panel LGG	Treatment Facilities	Medium	$498,000	$ -	$249,000	$249,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$498,000	$ -	$ -	$ -	49%	12%	22%	17%
E-04	Replacement of Circa 1998 Electrical Equipment	Treatment Facilities	Medium	$2,600,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$2,600,000	$ -	49%	12%	22%	17%
E-05	Replace MCC MCS and MCO	Treatment Facilities	High	$1,148,000	$ -	$574,000	$574,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$1,148,000	$ -	$ -	$ -	49%	12%	22%	17%
E-06	JWC Electrical Improvements Identified in Study (Include FWPS2 Transformer)	Treatment Facilities	High	$2,226,000	$ -	$ -	$ -	$889,000	$1,337,000	$ -	$ -	$ -	$ -	$ -	$2,226,000	$ -	$ -	$ -	49%	12%	22%	17%
R-01	Water Rights: S-55219	Water Rights	High	$100,000	$100,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$100,000	$ -	$ -	$ -	49%	12%	22%	17%
R-02	Water Rights: S-88506	Water Rights	Medium	$100,000	$ -	$ -	$ -	$100,000	$ -	$ -	$ -	$ -	$ -	$ -	$100,000	$ -	$ -	$ -	49%	12%	22%	17%
S-01	SCADA Program	SCADA	High	$4,952,000	$619,000	$619,000	$619,000	$619,000	$619,000	$619,000	$619,000	$619,000	$ -	$ -	$3,095,000	$1,857,000	$ -	$ -	49%	12%	22%	17%
T-01	Add Additional PRS Station	Treatment Facilities	High	$15,000	$15,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$15,000	$ -	$ -	$ -	49%	12%	22%	17%
T-02	Land Purchase for Future Intake	Land	Very High	$1,000,000	$1,000,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$1,000,000	$ -	$ -	$ -	49%	12%	22%	17%

No.	Project Name	Category	Priority	Total CIP Cost Estimate	FY 23/24	FY 24/25	FY 25/26	FY 26/27	FY 27/28	FY 28/29	FY 29/30	FY 30/31	FY 31/32	FY 32/33	Phase 1 (FY2024-2028)	Phase 2 (FY2029-2033)	Phase 3 (FY2034-2043)	Phase 4 (FY2044-2063)	Hillsboro Ownership	Forest Grove Ownership	Beaverton Ownership	TVWD Ownership
T-03	Hazard Mitigation Study	Treatment Facilities	High	$800,000	$300,000	$300,000	$200,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$800,000	$ -	$ -	$ -	49%	12%	22%	17%
T-04	New Intake or SHPP Upgrade	Treatment Facilities	High	$34,000,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$34,000,000	49%	12%	22%	17%
T-05	Initiate Study with USBOR for SHPP	Treatment Facilities	High	$200,000	$ -	$ -	$100,000	$100,000	$ -	$ -	$ -	$ -	$ -	$ -	$200,000	$ -	$ -	$ -	49%	12%	22%	17%
T-06	Hazard Mitigation Implementation	Treatment Facilities	Medium	$2,500,000	$ -	$ -	$ -	$ -	$ -	$500,000	$500,000	$500,000	$500,000	$500,000	$ -	$2,500,000	$ -	$ -	49%	12%	22%	17%
T-07	Replace Rapid Mix Facility	Treatment Facilities	Medium	$4,111,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$4,111,000	$ -	49%	12%	22%	17%
T-08	Replace Floc/Sed Basins A-C	Treatment Facilities	Medium	$24,789,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$24,789,000	49%	12%	22%	17%
T-09	CIMP: Add Hatches to Floc/Sed Basins	Treatment Facilities	Low	$137,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$137,000	$ -	49%	12%	22%	17%
T-10	Seismic: Structural - Strengthen Basins D-G Baffles	Treatment Facilities	High	$140,000	$ -	$140,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$140,000	$ -	$ -	$ -	49%	12%	22%	17%
T-11	Replace Filters 1-14	Treatment Facilities	Low	$41,855,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$41,855,000	49%	12%	22%	17%
T-12	Replace Existing Clearwell	Treatment Facilities	Medium	$37,246,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$37,246,000	$ -	49%	12%	22%	17%
T-13	FWPS 2 Seismic Improvements	Treatment Facilities	Low	$7,848,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$7,848,000	49%	12%	22%	17%
T-14	CIMP: Replace FW Pump 7	Treatment Facilities	Medium	$1,102,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$1,102,000	$ -	49%	12%	22%	17%
T-15	CIMP: Replace FW Pump 8 and BW Pump 3	Treatment Facilities	Medium	$1,310,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$1,310,000	$ -	49%	12%	22%	17%
T-16	CIMP: Replace FW Pump 9 and BW Pump 4	Treatment Facilities	Medium	$1,310,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$1,310,000	$ -	49%	12%	22%	17%
T-17	Construct Second Washwater Surge Basin	Treatment Facilities	Low	$2,491,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$2,491,000	49%	12%	22%	17%
T-18	Replace Chlorine Gas System	Treatment Facilities	Very High	$9,120,000	$1,000,000	$4,061,000	$4,060,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$9,120,000	$ -	$ -	$ -	49%	12%	22%	17%
T-19	Install OSHG Equipment into Hypo Facility	Treatment Facilities	Medium	$5,469,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$5,469,000	$ -	49%	12%	22%	17%
T-20	Construct New Chemical Building	Treatment Facilities	Medium	$8,596,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$8,596,000	$ -	49%	12%	22%	17%
T-21	Replace Operations and Maintenance Building	Treatment Facilities	High	$7,971,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$7,971,000	$ -	49%	12%	22%	17%
T-22	Hydropneumatic Surge Tank Liquefaction Mitigation	Treatment Facilities	Low	$112,000	$ -	$ -	$ -	$56,000	$56,000	$ -	$ -	$ -	$ -	$ -	$112,000	$ -	$ -	$ -	49%	12%	22%	17%
T-23	Update JWC CIP Master Plan Every 5 Years	Treatment Facilities	Low	$2,060,000	$ -	$ -	$ -	$ -	$140,000	$ -	$ -	$140,000	$500,000	$ -	$140,000	$640,000	$1,280,000	$ -	49%	12%	22%	17%
T-24	15,000-Gallon Diesel Fuel Tank	Treatment Facilities	High	$100,000	$100,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$100,000	$ -	$ -	$ -	68%	3%	12%	17%
T-25	New Alum Tank	Treatment Facilities	High	$200,000	$200,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$200,000	$ -	$ -	$ -	49%	12%	22%	17%
T-26	New Finished Water Pump 10 w/VFD	Treatment Facilities	Medium	$900,000	$ -	$ -	$ -	$450,000	$450,000	$ -	$ -	$ -	$ -	$ -	$900,000	$ -	$ -	$ -	49%	12%	22%	17%
T-27	Sedimentation Basins A/B/C Concrete	Treatment Facilities	Medium	$600,000	$ -	$ -	$200,000	$200,000	$200,000	$ -	$ -	$ -	$ -	$ -	$600,000	$ -	$ -	$ -	49%	12%	22%	17%
T-28	PAC Injection System Expansion	Treatment Facilities	Medium	$973,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$973,000	$ -	49%	12%	22%	17%

Table 9.7 provides a summary of the project by category and phase, and Figure 9.1 shows the Total CIP by project category.

Table 9.7 CIP Summary by Project Category

Project Category	Total CIP Cost	Phase 1 (FY2024-28)	Phase 2 (FY2029-2033)	Phase 3 (FY2034-43)	Phase 4 (FY2044-63)
Total Cost	**$277,845,000**	**$23,490,000**	**$18,141,000**	**$92,805,000**	**$140,983,000**
Land	$1,000,000	$1,000,000	$-	$-	$-
Treatment Facilities	$203,923,000	$17,245,000	$3,190,000	$72,505,000	$110,983,000
Transmission Lines	$67,770,000	$4,376,000	$13,094,000	$20,300,000	$30,000,000
Water Rights	$200,000	$200,000	$-	$-	$-
SCADA	$4,952,000	$3,095,000	$1,857,000	$-	$-
Annual Cost	**$-**	**$5,183,000**	**$3,628,000**	**$9,281,000**	**$7,049,000**

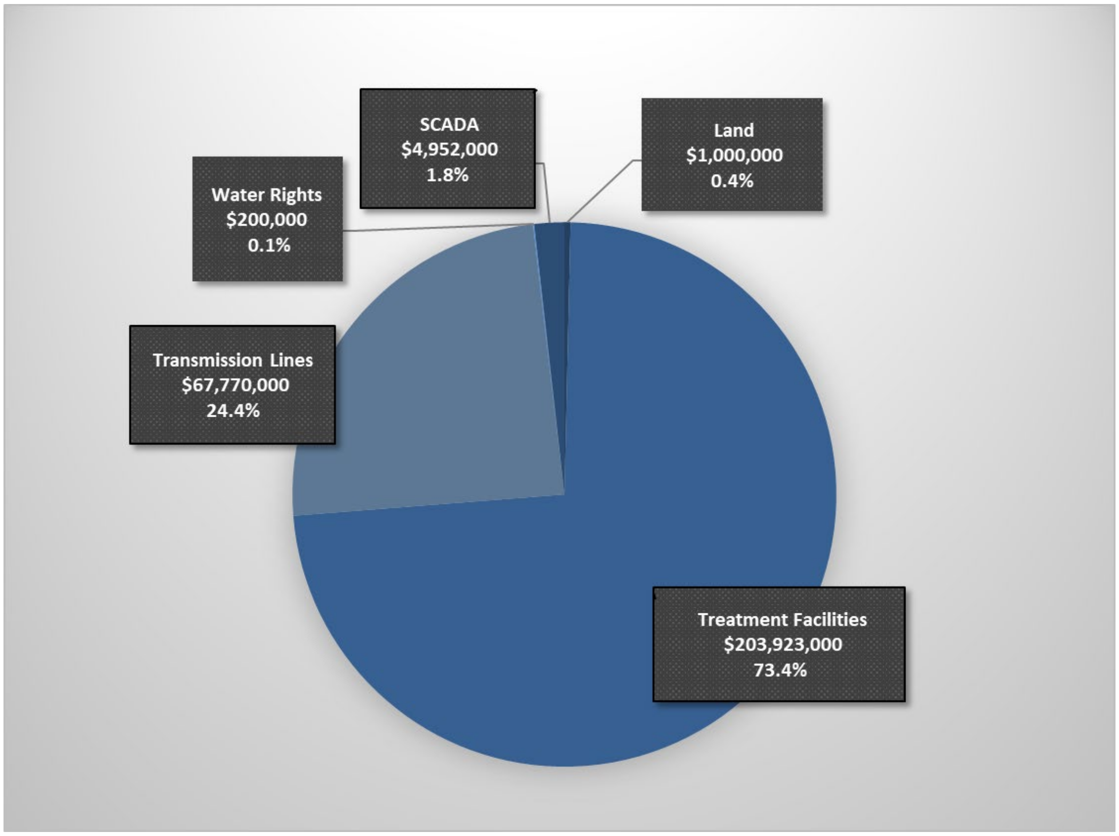


Figure 9.1 CIP Summary by Project Category

As shown in Table 9.7 and Figure 9.1, treatment facility projects make up approximately 73 percent of the total CIP costs. Transmission lines projects make up approximately 24 percent of the CIP with the remaining costs split between the SCADA, land, and water rights project categories. The recommended CIP averages $5.2M annually in Phase 1 and $3.6M annually in

Phase 2. The annual CIP costs then increase to $9.3M in Phase 3, and approximately $7M annually for Phase 4.

Figure 9.2 shows the CIP cost by phases. Phase 4, which spans 20 years from FY2044 to 2063, has most of the CIP costs with $140M. Phase 3, which spans 10 years from FY2034-2043, is $93M. Phase 1 is approximately $26M, and Phase 2 is approximately $18M. Phases 1 and 2 are five years each.

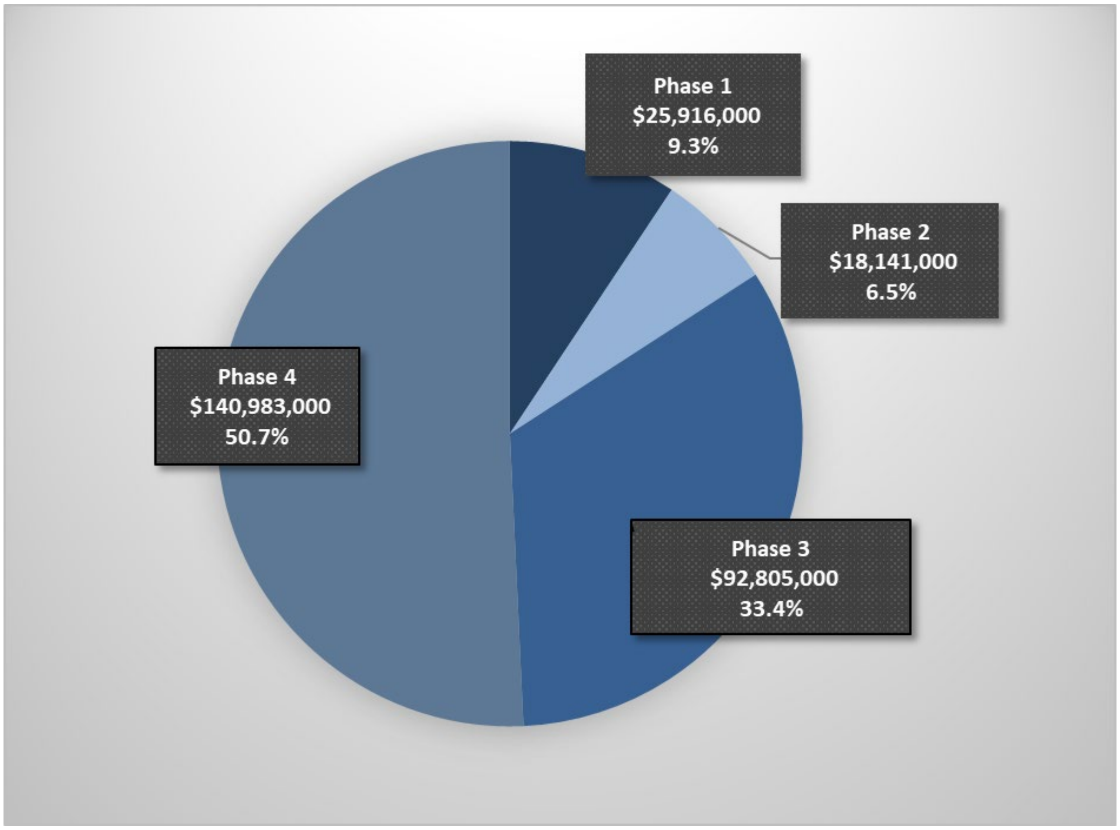


Figure 9.2 CIP Summary by Phase

Tables 9.8 and 9.9 summarize the total CIP cost by JWC partner and phase and annual CIP cost by JWC partner and phase, respectively.

Table 9.8 Total CIP Summary by JWC Partner

JWC Partner	Total CIP Cost	Phase 1 (FY2024-28)	Phase 2 (FY2029-33)	Phase 3 (FY2034-43)	Phase 4 (FY2044-63)
Total Cost	**$277,845,000**	**$25,916,000**	**$18,141,000**	**$92,805,000**	**$140,983,000**
Hillsboro	$136,056,000	$12,309,000	$8,911,000	$45,585,000	$69,251,000
Forest Grove	$32,640,000	$3,048,000	$2,133,000	$10,879,000	$16,580,000
TVWD	$61,260,000	$5,743,000	$4,002,000	$20,414,000	$31,101,000
Beaverton	$47,890,000	$4,816,000	$3,095,000	$15,927,000	$24,052,000

Table 9.9 Annual CIP Costs by JWC Partner

JWC Partner	Phase 1 (FY2024-28)	Phase 2 (FY2029-33)	Phase 3 (FY2034-43)	Phase 4 (FY2044-63)
Annual Cost	**$5,183,000**	**$3,628,000**	**$9,281,000**	**$7,049,000**
Hillsboro	$2,462,000	$1,782,000	$4,558,000	$3,463,000
Forest Grove	$610,000	$427,000	$1,088,000	$829,000
TVWD	$1,149,000	$800,000	$2,041,000	$1,555,000
Beaverton	$963,000	$619,000	$1,593,000	$1,203,000

After the CIP was established, each project was evaluated for funding eligibility. Table 9.10 lists the projects most eligible for funding and the potential funding sources. Details on each of these funding sources and how they are applicable to JWC projects can be found in Appendix N.

Table 9.10 Summary of CIP Projects and Funding Opportunities

No.	Project Name	Timing	Low-Interest Loan: EPA; WIFIA	Low-Interest Loan: Oregon IFA SDWRLF	Legislative: Community Project Funding	FEMA: BRIC	FEMA: FMA	FEMA: HMGP	USBOR: Water & Energy Efficiency Grants	USBOR: Small-Scale Water Efficiency Projects	DOE: Energy Efficiency and Conservation Block Grant Program	OWRD: Water Project Grants & Loans	OEM: SPIRE
S-01	SCADA Program	2024 to 2031	√	√	√				√		√	√	
T-24	15,000-Gallon Diesel Fuel Tank$^{(1)}$	2024	√	√	√								√
T-03	Hazard Mitigation Study	2024 to 2026	√	√	√	√		√					
T-18	Replace Chlorine Gas System	2024 to 2026	√	√	√			√					
T-05	Initiate Study with USBOR for SHPP$^{(2)}$	2026 to 2027											
D-09	Beaverton and TVWD Billing Meters	2027	√	√						√	√		
D-03, D-08	Seismic Valves on N-S Intertie and Fern Hill Force Mains/Replace/Improve Transmission Lines in Critical High-Hazard Areas (20-year CIP)$^{(3)}$	2029 to 2043	√	√	√	√		√					
T-06	Hazard Mitigation Implementation	2029 to 2033	√	√	√			√					
T-07, T-21	Replace Rapid Mix Facility/ Replace O&M Building$^{(4)}$	2034 to 2036	√	√	√			√			√		
T-19	Install OSHG Equipment into Hypo Facility$^{(5)}$	2037 to 2039	√	√	√	√		√					
T-20	Construct New Chemical Building$^{(6)}$	2037 to 2039	√	√	√	√		√			√		
T-04	New Intake or SHPP Upgrade	Beyond 2044	√	√	√	√	√	√					

Notes:

Abbreviations: BRIC – Building Resilient Infrastructure and Communities; DOE – Department of Energy; EPA – U.S. Environmental Protection Agency; FEMA – Federal Emergency Management Act; FMA – Flood Mitigation Assistance; HMGP – Hazard Mitigation Grant Program; IFA – Business Oregon Infrastructure Finance Authority; OEM – Oregon Department of Energy Management; OWRD – Oregon Water Resources Department; SDWRLF – Safe Drinking Water State Revolving Loan Fund; SPIRE – State Preparedness and Incident Response Equipment; WIFIA – Water Infrastructure Finance Innovation Act.

(1) Project cost less than JWC funding threshold of $1 million, but favorable funding opportunity is available.

(2) JWC should consider approaching USBOR for funding assistance with study.

(3) Due to length of project, D-08 could be phased and combined with D-03 to maximize available funding opportunities.

(4) Because projects are so closely aligned, projects may be combined to maximize funding opportunity.

(5) Project may be funded alone, but JWC should consider combining project with T-20 to maximize funding opportunity.

(6) Project may be funded alone, but JWC should consider combining project with T-19 to maximize funding opportunity.