WHERE THE ROGUE RIVER RUNS

GRANTS PASS

OREGON

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Water Treatment Plant Facility Plan Update

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Engineers/Planners

MWH

City of Grants Pass

January 2014

WATER TREATMENT PLANT FACILITY PLAN UPDATE

FOR

CITY OF GRANT PASS

JANUARY 2014

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RENEWS 6-30-15

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ACKNOWLEDGEMENTS

The following City of Grants Pass staff contributed to the development of this Water Treatment Plant Facility Plan Update.

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- Jim Williams
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- Darin Fowler, Mayor
- Lily Morgan
- Rick Riker
- Dan DeYoung
- Mark Gatlin
- Jim Goodwin

ACRONYMS AND ABBREVIATIONS

The following is a list of definitions of acronyms and abbreviations.

°C	degrees Celsius
°F	degrees Fahrenheit
ACH	aluminum chlorohydrate
ADA	Americans with Disabilities Act
ADD	average day demand
$Al_2(SO_4)$	aluminum sulfate or alum
AWWA	American Water Works Association
BF	ballasted flocculation
$CaCO_3$	calcium carbonate
$Ca(OH)_2$	calcium hydroxide
$Ca(S_2O_3)$	calcium thiosulfate
CFE	combined filter effluent
CFR	Code of Federal Regulations
CIP	capital improvement program
City	City of Grants Pass
Cl_2	free chlorine
CMU	concrete masonry unit
CO_2	carbon dioxide
Cr+6	hexavalent chromium
CT	disinfectant concentration multiplied by the contact time
D/DBP	Disinfectants and Disinfection Byproducts
DBP	disinfection by-products
E. coli	*Escherichia coli*
EDC	endocrine disruptor
EPA	Environmental Protection Agency (United States)
ESWTR	Enhanced Surface Water Treatment Rule
FBRR	Filter Backwash Recycling Rule
fps	feet per second
FRP	fiberglass reinforced plastic
ft	foot or feet
FTE	full-time employee
FTW	filter-to-waste
FWP	finished water pipeline
FWPS	finished water pump station
g/L	grams per liter
GAC	granular activated carbon
gal	gallon or gallons
gal/sf	gallons per square foot
GIS	geographic information system
gph	gallons per hour

gpm	gallons per minute
gpm/sf	gallons per minute per square foot
HAA5	haloacetic acids – D/DBP rule
HDPE	high-density polyethylene
HGL	hydraulic grade line
HI	Hydraulic Institute
HSPS	high service pump station
HP	horsepower
HRT	hydraulic retention time
HVAC	heating, ventilation, and air conditioning
in.	inch or inches
IOC	inorganic compound
k	thousands
$KMnO_4$	potassium permanganate
kW	kilowatt
lb/sf/day	pounds per square foot per day
LCR	Lead and Copper Rule
LOX	liquid oxygen
LRAA	Locational Running Annual Average
LT2	Long-Term 2
LT2ESWTR	Long-Term 2 Enhanced Surface Water Treatment Rule
M	million
mA	milliampere
MCL	maximum contaminant level
MDD	maximum day demand or peak day demand
MG	million gallon
mg/L	milligrams per liter
mgd	million gallons per day
MIB	2-Methylisoborneol
min	minute or minutes
mL	milliliter
mm	millimeter or millimeters
MRDL	maximum residual disinfectant level
MSA	Murray, Smith and Associates, Inc.
MWH	MWH Americas, Inc.
N	nitrogen
Na_2CO_3	soda ash
NaOCl	sodium hypochlorite
NaOH	sodium hydroxide
NPDES	National Pollutant Discharge Elimination System
NTU	Nephelometric Turbidity Units
O_3	ozone
OAR	Oregon Administrative Rules
OCl	hypochlorite
OHA	Oregon Health Authority

OSHA	Occupational Safety and Health Administration
PAC	powdered activated carbon
PACl	polyaluminum chloride
pCi/L	picocuries per liter
PCP	personal care product
pH	measure of the acidic or basicity of an aqueous solution
phAC	pharmaceuticals
ppb	parts per billion
ppd	pounds per day
ppm	parts per million
ppt	parts per trillion
psi	pounds per square inch
psig	pounds per square inch gauge
PVC	polyvinyl chloride
RAA	Running Annual Average
RO	reverse osmosis
ROW	right-of-way
rpm	revolutions per minute
RWP	raw water pipeline
SCADA	Supervisory Control and Data Acquisition
SOC	synthetic organic compound
ST1DBPR	Stage 1 Disinfectants and Disinfection Byproducts Rule
ST2DBPR	Stage 2 Disinfectants and Disinfection Byproducts Rule
State	State of Oregon
SWTR	Surface Water Treatment Rule
T or T_{10}	"effective" detention time through a section of the plant
TC	total coliform
TCR	Total Coliform Rule
TDH	total dynamic head
THM	trihalomethanes
TOC	total organic carbon
TON	threshold odor number
TSS	total suspended solids
TTHM	total trihalomethanes
UCMR	Unregulated Contaminants Monitoring Rule
UL	Underwriters Laboratories
USGS	United States Geological Survey
UV	ultraviolet
UVA	ultraviolet A
V	volt
VDC	volts direct current
VFD	variable frequency drive
VOC	volatile organic compound
WTPFP	Water Treatment Plant Facility Plan
WTP	water treatment plant

WW	washwater
WWW	waste washwater
yr	year

EXECUTIVE SUMMARY

EXECUTIVE SUMMARY

This report presents an update to the 2004 Water Treatment Plant Facility Plan (WTPFP) for the City of Grants Pass (City). The 2004 WTPFP provided guidance for improving the City's water treatment plant (WTP) and recommended a two-tiered capital improvement program (CIP). The City implemented a number of the recommended improvements which included addressing reliability and redundancy shortfalls and performing critical structural crack repairs in process basins.

Despite these improvements, conditions at the WTP have deteriorated since the 2004 WTPFP was created. In order to continue to reliably produce water for the community, a significant decision needs to be made in regards to the existing treatment facility. This WTPFP update provides the City with a sound basis for making the key decision: proceed with further major capital investments to maintain the existing facility or proceed with planning, design and construction of a new WTP to replace the aging facility. Both alternatives require immediate capital expenditures from the City to secure the water system's reliability. The final decision will lay the foundation for more than a hundred years of water system operations and will need to balance the economic, social, and environmental needs of the City.

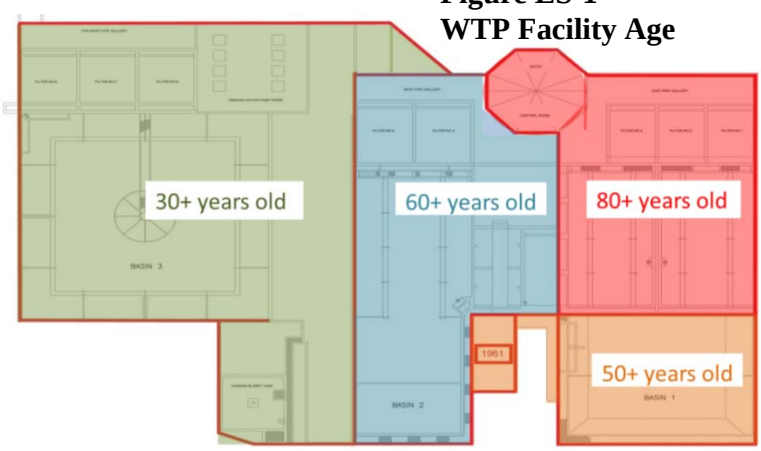
The primary objectives of this WTPFP update included:

- Evaluate the recent performance of the WTP in terms of quality and capacity
- Update the impacts of current drinking water regulations as they affect current and future treatment requirements
- Evaluation and documentation of the existing condition and remaining useful life of the WTP's structural systems
- Incorporation of recently updated water system demand projections to help identify potential WTP capacity deficiencies and the need for development of expanded capacity
- Evaluation of alternative approaches for maintaining the existing WTP and providing expanded capacity
- Evaluation of siting and construction of a new WTP
- Develop a CIP implementation plan based on community stakeholder input and triple bottom line analyses of existing and new facility improvement alternatives

Water Treatment Plant Overview

The Grants Pass WTP, located at 821 Southeast "M" Street, was originally built in 1931 and has undergone several upgrades and expansions to serve a growing population and to meet more stringent treatment standards. Capacity upgrades were completed in 1950, 1961, and 1983, as illustrated in **Figure ES-1**. The plant's current hydraulic capacity is approximately 20 mgd.

Figure ES-1
WTP Facility Age



Water Treatment Plant Condition

Several structures at the WTP continue to show increasing signs of deterioration as many parts of the WTP have reached or exceeded their expected service life. As highlighted in the pictures on this page, the deterioration includes:

- Exposed rebar and concrete failure in sections of the clearwell.
- Spalling and cracking concrete in older primary process components of the WTP.
- Failure of submerged structural elements.

All of these elements are critical in supplying a reliable quantity and quality of drinking water to the citizens of Grants Pass.

A seismic and structural review of the Grants Pass WTP was completed in 2011 in response to the observed structural deteriorations. The review concluded that the WTP is at a high seismic risk and **is susceptible to collapse in a strong earthquake**. A planning-level project cost to address deficiencies observed during the review was estimated to be approximately $8.5 million. While these structural improvements would reduce the overall seismic vulnerability of the WTP, they do not improve the facilities to current building code standards for seismic events, and they do little to address the declining condition of the aging facility and would not increase WTP production capacity.

Spalled concrete and exposed rebar

Exposed rebar in clearwell beam

Vertical cracking in basin wall

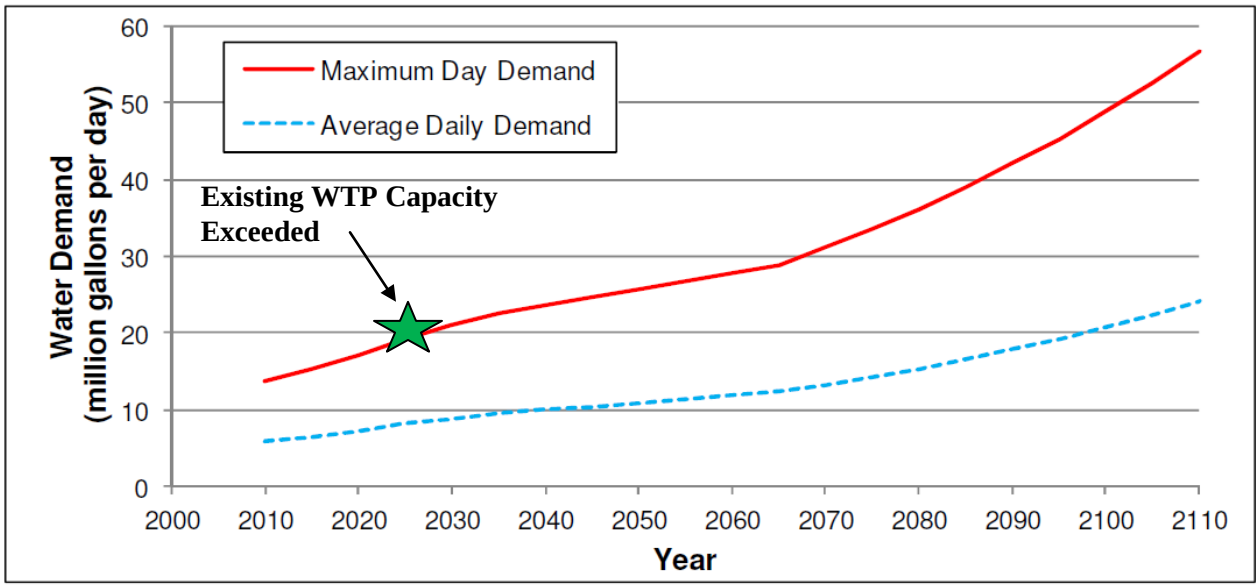
In February 2013, the Oregon Resilience Plan (Plan) was completed, highlighting the real risk of a major Cascadia Subduction Zone earthquake with a magnitude of 9.0. One of the key recommendations of the Plan is the completion of comprehensive assessment and mitigation plans for critical water system infrastructure. **For the City of Grants Pass, the WTP, as the City's sole source of water supply with no emergency backup, is the most critical facility in the water system.** The age and condition of the WTP, as described herein, emphasizes how vulnerable this facility is to catastrophic damage in a major earthquake.

Water Demand Projections and Capacity Needs

The design flow for WTP capacity is the maximum day demand (MDD) for water utilities that have adequate distribution system storage. Per regulatory requirements, the development of water demand projections consider the existing service area, future service areas, and trending of

historical population and water demand information. As illustrated in **Figure ES-2**, the MDD is projected to exceed the existing WTP capacity between 2025 and 2030, and will reach approximately 30 mgd by 2065. While continued reduction in water use through conservation and increased efficiency may delay the need for expanded capacity, water demands will continue to increase over time as the City's population grows, ultimately requiring expanded water treatment capacity.

Figure ES-2
Water Demand Projections



Capital Improvement Alternatives Overview

Five capital improvement alternatives were developed to represent a full range of potential space, cost, and risk scenarios that address the identified WTP deficiencies and promote reliable, long-term source of supply from the Rogue River. The alternatives are:

Alternative 1: Existing WTP Upgrade, Maximize Reuse of Existing Facilities

Alternative 2: Existing WTP Upgrade, Phased Replacement of Facilities

Alternative 3: Construct a New WTP with Consolidated Footprint

Alternative 4: Construct a New WTP with Large Footprint

Alternative 5: Construct a New WTP with Consolidated Footprint on Property already owned by the City

Capital Improvements Program Recommendation

An Advisory Committee of community leaders and City Council members was assembled to assist in the evaluation and recommendation of a preferred alternative from those presented above. City Public Works employees integral to the project also participated to offer information about operational impacts, zoning and land use implications, and necessary steps in the City approval process.

A series of four workshops was conducted over a three-month period with the Advisory Committee using an independent facilitator from the Grants Pass community. The Advisory Committee evaluated each alternative considering its economic, social, and environmental impacts.

For the benefit of the Advisory Committee, a list of suggested criteria was developed from similar projects to evaluate the alternatives. The committee then modified and finalized the criteria, establishing appropriate weighting for each through group discussion. **The members of the Advisory Committee independently scored the alternatives and Alternatives 3 and 4 - building a new WTP at a site to be determined - scored the highest**. These alternatives were selected by the Advisory Committee because they provide the City with a seismically secure, reliable WTP, with a treatment process able to provide improved water quality for existing and future generations, all at a cost to the community that is comparable to continuing to invest in the existing WTP.

Workshop results and scoring were presented to the City Council during its August 5, 2013 Workshop and then discussed further during its September 9, 2013 Workshop. In reviewing the materials developed and scoring performed, **the Council directed the completion of this Facility Plan Update with the recommendation to move forward in the planning process to construct a new WTP at a site to be determined**.

Capital Improvement Program Implementation Plan

The conceptual project cost to construct a new WTP is estimated to be approximately $56 million, with an accuracy range of –30 percent to + 50 percent. **It is recommended that the City establish a capital budget for this project which reflects this estimate and the level of uncertainty and risk associated with the current level of project definition**. This budget should be updated and refined over time as the implementation plan progresses and planning and design uncertainties are addressed.

The City should proceed with the processes necessary to construct the new WTP as quickly as possible to avoid extensive investments in the existing plant that are critical to ensuring the plant can continue to reliably meet water quality regulations. Delays in implementation would require the City to remain reliant upon the existing WTP. This presents not only significant structural and seismic reliability risks, but other production reliability and redundancy issues as highlighted in this WTPFP Update. The required capital investment in the existing WTP to mitigate these risks will increase over time, and represent stranded investments once the new WTP is constructed.

The recommended schedule to proceed with the planning, design and construction of the new WTP is presented in **Figure ES-3**. It is possible to have a new WTP online by the middle of 2019 using a traditional design-bid-build project delivery approach. **Table ES-1** presents a summary of anticipated capital expenditures (project costs) for the next 10 years to implement a new WTP based on this Implementation Schedule. While WTP construction would be completed in year 6, additional expenditure activities such as decommissioning of the existing WTP, reconfiguration of the WTP site, and the warranty period for the new WTP, will extend through the remainder of the 10-year timeframe.

Figure ES-3
Project Implementation Schedule

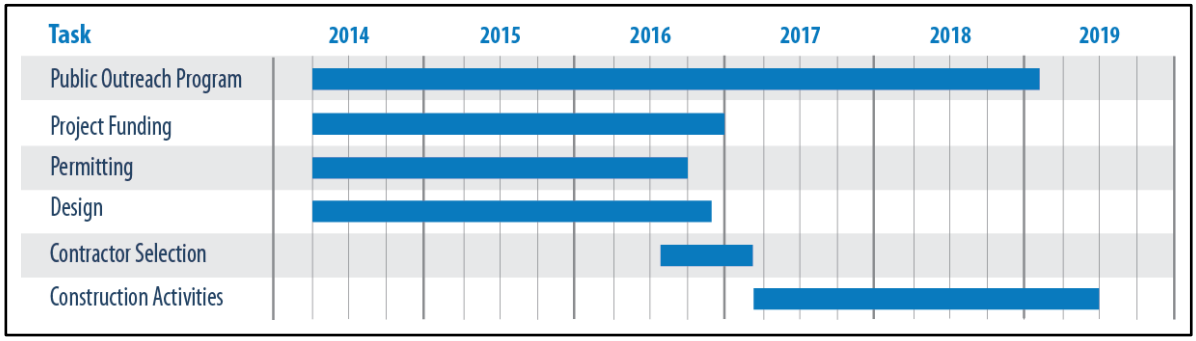


Table ES-1
Recommended Capital Improvement Program Summary

Capital Project	Capital Expenditure
New Water Treatment Plant Implementation	
Pilot Plant Study	$500,000
Siting Study and Property Acquisition	$1,300,000
Funding Study and Rate Impact Study	$200,000
Project Implementation Approach and Procurement Strategy	$50,000
Public Information/Involvement	$250,000
Permitting and Land-Use Approvals	$200,000
Preliminary Design	$1,000,000
Final Design	$4,000,000
Bidding and Award	$250,000
Construction	$47,200,000
Post-Construction and Warranty Period	$200,000
Existing Water Treatment Plant Investments	
Emergency Response Plan	$50,000
Decommission and Demolition of Existing Plant	$1,000,000
Total Anticipated Expenditures (2013 dollars)	**$56,200,000**

Project Initiation Activities

To bring a new WTP on-line by 2019, the City will be required to accomplish the following tasks during the first year of planning, beginning with the FY2014-2015 budget:

- Develop a funding strategy.
- Prepare an Emergency Response Plan for the potential failure of the existing WTP.
- Select a site for the new WTP.
- Conduct a year-long pilot plant study to evaluate clarification, intermediate disinfection and high-rate filtration process alternatives.
- Confirm the project schedule and project delivery strategy.
- Plan and implement a public outreach program.
- Develop a permitting and regulatory approval plan.

It is anticipated that the City will need to allocate approximately $1 million to complete these activities. Once significant progress has been made on each of these tasks, the detailed design phase may begin. If the City does not proceed with the proposed implementation plan and schedule to construct a new WTP, then funds should immediately be allocated and expended to complete critical structural and seismic repairs at the existing WTP. These improvements, including approximately $8.5 million dollars in structural upgrades, would be necessary to reduce the risk of failure of essential treatment process components in the interim. **Critical early planning activities should begin in the next fiscal year to avoid additional stranded capital investments in the existing WTP**.

CHAPTER 1

CHAPTER 1
INTRODUCTION AND BACKGROUND

Purpose

This report presents an update to the 2004 WTPFP for the City. The 2004 WTPFP provided guidance for improving this major element of the City's water system and recommended a two-tiered CIP. The main elements of the CIP were developed after a review and evaluation of historical plant performance and regulatory requirements. Many of the recommended improvements have been completed since 2004. The objectives of this WTPFP update include:

- Update the impacts of current drinking water regulations as they affect current and future treatment requirements.
- Update the capacity evaluation of the WTP, incorporating facility improvements and operational changes that have been implemented since 2004.
- Review information presented in recent structural evaluations and additional tests to help determine the remaining useful life of the WTP's structural systems.
- Visually inspect and review equipment in terms of age, condition, and code compliance to assess the remaining useful life of electrical and mechanical equipment.
- Incorporate recently updated water system demand projections to help identify potential plant capacity deficiencies. These demands were developed to assist in the City's water rights extension process and will be adopted into upcoming water system master planning documents.
- Evaluate alternative improvements to address existing and potential future WTP deficiencies.
- Feasibility evaluation for siting and construction of a new WTP.
- Develop planning-level cost estimates associated with both existing and new WTP project alternatives.
- Assist in selecting a capital improvements implementation plan based on input from an Advisory Committee of community leaders and City Council members and an analysis of existing and new facility improvement alternatives.

Project Background

The WTP uses chemical and physical processes to treat water from the Rogue River to produce high-quality drinking water and is the City's sole source of potable water. The original WTP facilities were constructed in 1931 with subsequent expansion projects in 1950, 1961, and 1983.

Due to observed and on-going deterioration in some older structural elements of the WTP, a seismic and structural review of the Grants Pass WTP was completed in 2011. A review of geotechnical studies conducted at the plant site showed that ground shaking and slope

stability along the Rogue River bluff are the two most significant seismic geotechnical risks. A review of the construction documents of the plant shows that, overall, the structures appear to have been designed and detailed prior to consideration of seismic loads. Given this lack of seismic design consideration, the plant is judged to have a high seismic risk and is susceptible to collapse in a strong earthquake.

A planning-level project cost to retrofit the various structural deficiencies observed during the review was estimated to be approximately $8.5 million. These structural studies and observations led the City to initiate this update to the WTPFP to help guide the planning of the City's water treatment and supply system and to help the City prioritize improvements over the next 20 years. This updated WTPFP evaluates the advantages and disadvantages of continuing to invest in this older, structurally deficient facility considering that it may have limited remaining useful life.

Water Treatment Plant Overview

The Grants Pass WTP, located at 821 Southeast "M" Street, was originally built in 1931 with a single sedimentation basin and three filters for a design capacity of approximately 3.5 mgd. The plant has undergone several upgrades and expansions to serve a growing population and to meet more stringent treatment standards. Capacity upgrades were completed in 1950, 1961, and 1983, and the plant has received numerous process and safety upgrades over the past two decades as well. The plant's current hydraulic capacity is approximately 20 mgd, but the plant cannot operate at this rate throughout the year due to process and regulatory compliance constraints during the colder months of the year. These constraints have yet to affect the plant's ability to produce high-quality water while meeting the City's water demands. The 1983 expansion required extensive internal remodeling of the original building, while preserving its current listing as a Historic Landmark by the Grants Pass Historic Building and Sites Commission and the American Water Works Association's (AWWA) National Historic Water Landmarks.

Raw Water Supply

The plant draws water from the adjacent intake on the Rogue River. The City has been drawing water from the Rogue since 1888 and currently has a total water right of 82 cubic feet per second (CFS) or 53 mgd. The river is prone to turbidity events and yearly fluctuations in temperature and pH which create seasonal challenges to plant operations. The river flow and quality are also influenced by upstream dam operations, most notably the Lost Creek Reservoir. In 2010, the Gold Ray Dam on the Rogue River was removed and this has created additional challenges to plant operations, including increased sediment and turbidity which among other things, has negatively affected the performance of the intake and screen cleaning system.

Facilities and Processes

The WTP is operated and rated as a conventional filtration plant, although it lacks flocculation prior to sedimentation in its basins. Liquid residuals, including dirty backwash water and filter-to-waste water, are transferred to the old mill pond, located across the street from the plant which overflows to Skunk Creek. The majority of plant solids, which collect in the sedimentation basins, are now handled on-site, but were previously discharged to the old mill pond along with the backwash water and filter-to-waste water.

Figure 1-1 is a photographic overview of the City's Water Treatment System and Figure 1-2 provides a plan-view layout of the WTP's current configuration. Figure 1-3 is a Process Flow Schematic of the plant indicating key processes and chemical addition points. Major facilities and structures at the Grants Pass WTP include:

- Raw water intake and screening facility.
- Raw water pumping station which has four pumps, all with 75 HP motors and two with variable frequency drives (VFDs), a flow meter, and 36-inch diameter static mixer.
- One mixing basin, currently operating as a flow-through structure without mixing, servicing basins 1 and 2.
- Three sedimentation basins with a total surface area of 18,800 square feet and total volume of 1,835,300 gallons.
- Eight dual media gravity filters with 30 inches of media depth and a total of 2,493 square feet of surface area.
- A 433,000 gallon baffled clearwell. The clearwell's volume of 433,000 gallons represents the maximum volume at the overflow level. The actual operating volume varies from 362,000 gallons to 400,000 gallons.
- Two backwash pumps with VFDs, 16-inch diameter backwash supply pipeline and flow meter.
- A high service pumping station which has six pumps, one constant-speed pump with 300 HP motor, two constant-speed pumps with 250 HP motors, two pumps with VFD 250 HP motors, and one pump with VFD 200 HP motor.
- One 36-inch diameter finished water transmission pipeline with flow meter.
- One hydropneumatic surge tank with a volume of 11,300 gallons located on the finished water discharge.
- Chemical storage, metering, and rapid mixing systems for liquid alum, liquid proprietary coagulant, liquid sodium hypochlorite, and dry polymer. ACH is used as the primary coagulant, alum is used as a supplemental coagulant and to aid in pH adjustment, and filter aid polymer is added to the basin effluent to improve filter performance. Disinfection is achieved through both pre- and post-chlorination by addition of sodium hypochlorite.
- One 116,000-gallon equalization basin for backwash wastewater and filter-to-waste water.

- Equalization basin pumping station with three pumps, two pumps with 30 HP each with a combined capacity of 2,100 gpm at a TDH of 42 feet, and one pump with 60 HP motor rated at 1,750 gpm at a TDH of 60 feet.
- One residual solids lagoon, called the old mill pond, which discharges decant into Skunk Creek and eventually into the Rogue River.
- The old powdered activated carbon slurry tank was re-purposed as a solids conditioning tank to receive residual solids from sedimentation basins. After conditioning with polymer, the solids are pumped into geomembrane bags or "geobags" for on-site dewatering and hauled to off-site disposal.

N

0 50 100

FULL SIZE: 1" = 50'
HALF SIZE: 1" = 100'

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REV	DATE	BY	DESCRIPTION

SCALE: AS NOTED

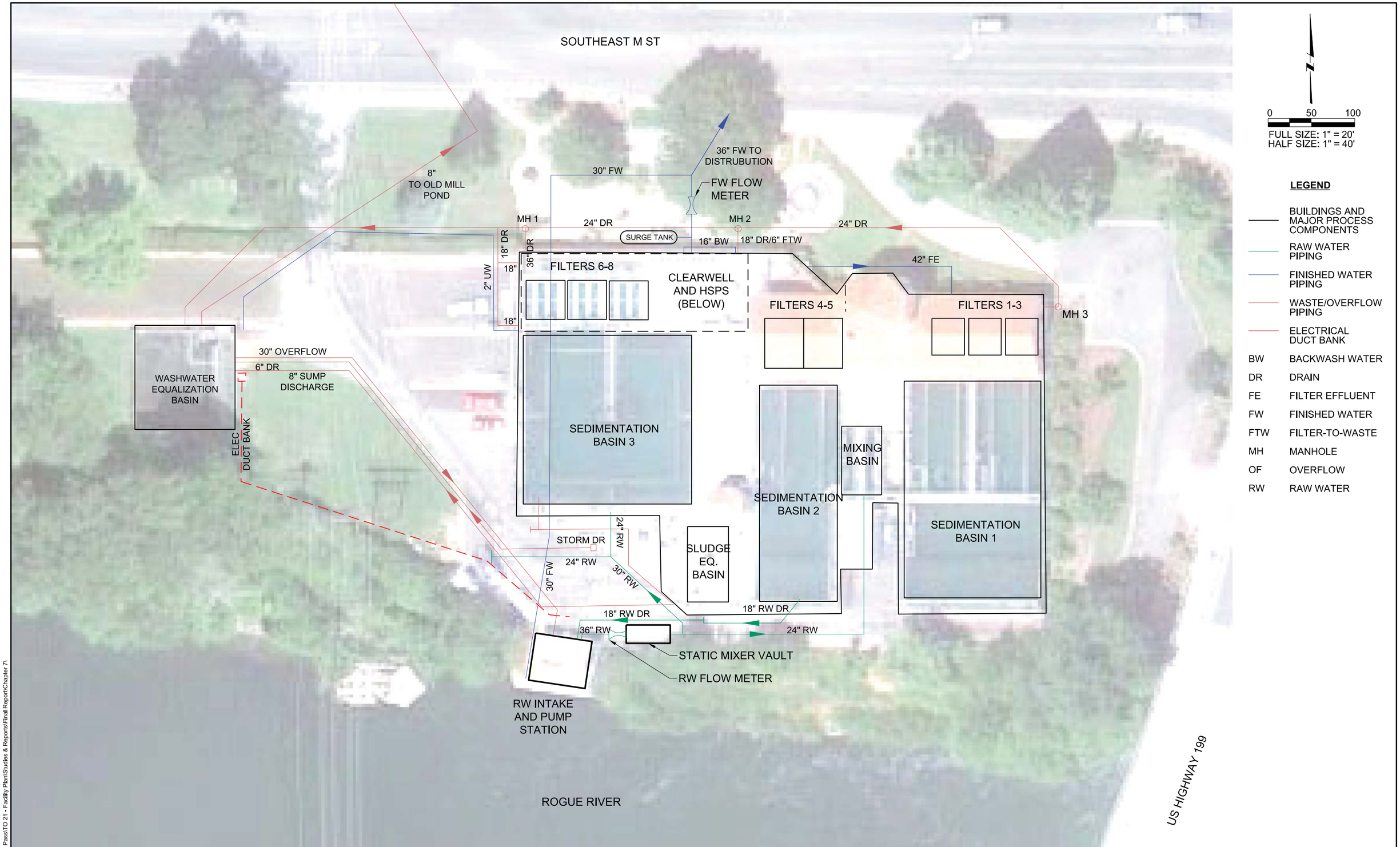
DESIGNED A. NISHIHARA
DRAWN A. ORR
CHECKED P. KREFT

MSA

MWH®

City of Grants Pass

WATER TREATMENT PLANT FACILITY PLAN UPDATE
FIGURE 1-1
EXISTING SYSTEM OVERVIEW



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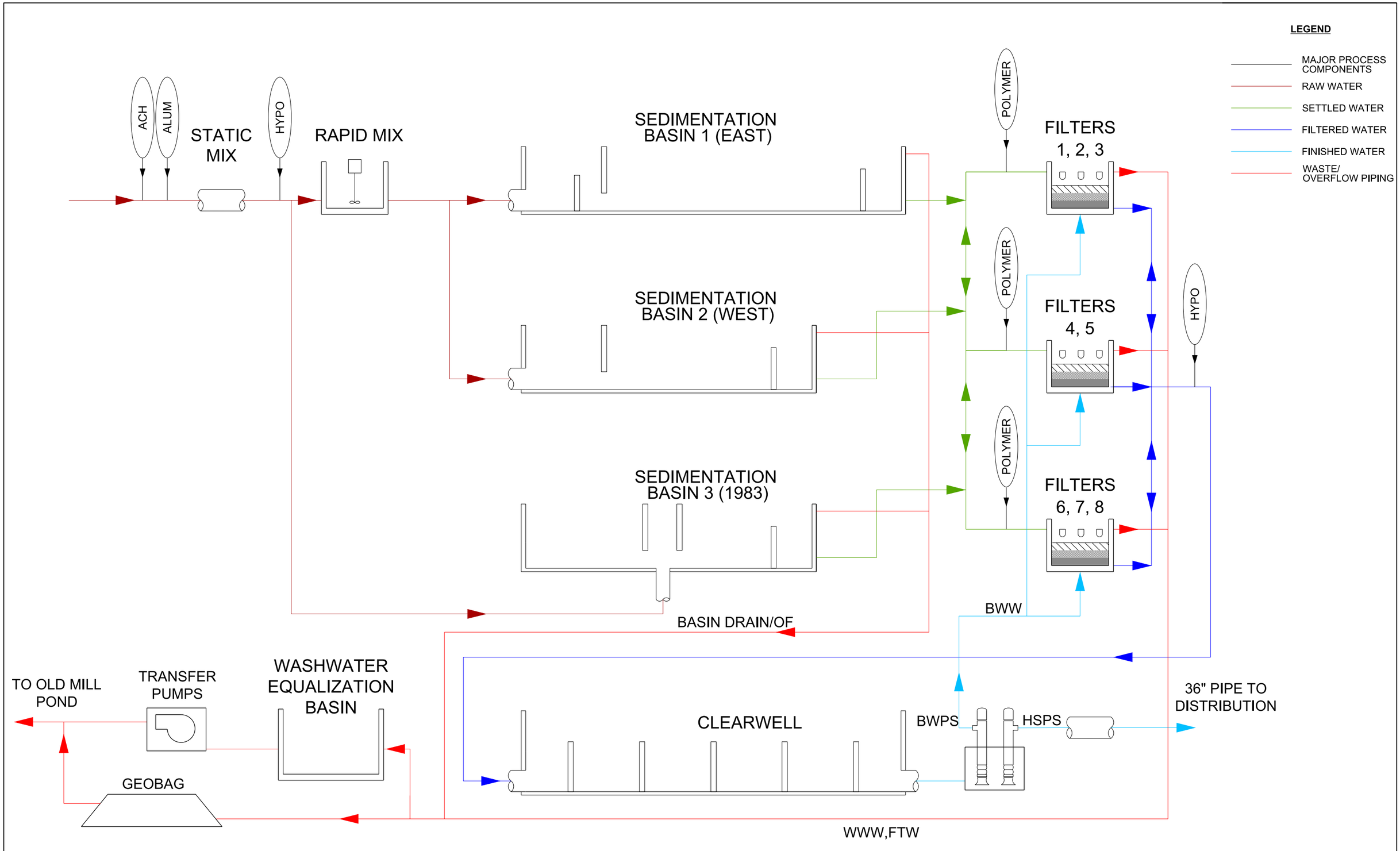
WATER TREATMENT PLANT FACILITY PLAN UPDATE
FIGURE 1-2
PLAN VIEW LAYOUT



REV	DATE	BY	DESCRIPTION	SCALE						
				NO SCALE		DESIGNED A. NISHIHARA DRAWN A. ORR CHECKED P. KREFT	MSA	MWH	City of Grants Pass	WATER TREATMENT PLANT FACILITY PLAN UPDATE FIGURE 1-3 EXISTING SYSTEM PROCESS FLOW SCHEMATIC	PAGE 1-6

The operations building includes a water quality laboratory for treatment process monitoring and control, the plant's electrical distribution equipment, main control board, and other instrumentation and control equipment. The operations building also has office and administrative spaces, a lunchroom, workshop, and meeting area.

The plant typically operates between 8 and 24 hours per day, depending on system demands. During the peak demand months of July through September, the plant is operated for up to 24 hours per day to meet peak day demands. The plant is staffed at all times when operating and employs six full-time employees (FTE) and five seasonal, part-time employees.

Plant Improvements Since 2004

The 2004 WTPFP recommended a two-tiered CIP to improve plant performance to meet regulatory requirements and improve overall plant operations and safety. Major improvements made at the WTP since the 2004 WTPFP include:

- Screening improvements at the Rogue River intake to meet fish protection criteria.
- Addition of VFDs to the raw water pumps to improve plant operating flexibility.
- Replacement of filter underdrains and media to improve plant performance.
- Addition of solids handling and dewatering facilities to reduce the volume of solids and liquids being discharged to the old mill pond across the street from the WTP; "Geobags" are now used to handle solids for dewatering.
- Incorporation of a new coagulation chemical scheme which has eliminated the need to add lime for pH adjustment; hence, the lime system has been demolished.
- Addition of a second filter backwash pump to improve plant reliability.
- Addition of a standby generator at the WTP to improve plant reliability and reduce vulnerability during power outages; to be completed in the spring of 2014.

A project involving solids removal systems in the sedimentation basins was investigated in 2009 and this work was deferred by the City due to high costs. The high cost was due to structural and seismic retrofitting required as part of the installation of the equipment.

Summary

Chapter 1 establishes the purpose of this Facility Plan update and provides background on activities leading up to its development. Subsequent chapters will review various aspects of the WTP's condition and performance. The evaluation of the existing plant includes a performance evaluation, regulatory review, capacity review, and facilities review. Each review is summarized in separate sections of this report. These reviews and analyses document potential improvements at the existing WTP which may be required for a number of reasons including maintaining existing capacity, increasing capacity, optimizing performance, meeting future drinking water regulations, extending remaining useful facility life, and improving safety and operational efficiency. Proposed WTP improvement

alternatives include estimated project costs and alternatives that consider construction of a new WTP at a site other than the existing WTP site.

The evaluation and final recommendation of the preferred capital improvement alternative was accomplished using a triple-bottom-line analysis. The City assembled an Advisory Committee of community leaders and City Council members to assist in the evaluation and recommendation of a preferred alternative. The Advisory Committee convened for four separate workshops during the summer of 2013 to review and discuss the alternatives before selecting a preferred alternative.

CHAPTER 2
HISTORICAL PLANT PERFORMANCE

Introduction

Historic operating data for the Grants Pass WTP are reviewed and analyzed in this chapter. The purpose of this data review is to evaluate the existing WTP processes for capacity, operational efficiency, and regulatory compliance. Data collected and reviewed from 2004 to 2011, the period after completion of the 2004 WTPFP, included: plant flow and production information; selected raw, finished and distribution system water quality parameters; basin performance; chemical usage data; and overall filter performance indicators. As highlighted in Chapter 1, a number of recommended improvements in the 2004 WTPFP have been completed at the plant and have had a beneficial impact on plant performance.

Water Treatment Plant Production

The Grants Pass WTP measures and records raw and finished water flows through the plant on a daily basis. Raw water flow is measured using a differential pressure type (Venturi) flow meter located on the influent line prior to chemical addition. Finished water flow is measured using a Venturi flow meter located on the WTP effluent line just downstream of the HSPS. Filter backwash flow is measured in the backwash supply line. FTW flows are discharged upstream of the filter effluent flow meters, and therefore have not been historically measured or recorded since the installation of the FTW line in 1997. The duration of FTW after filters are backwashed usually only lasts for a few minutes and has limited impact on overall plant flow and performance.

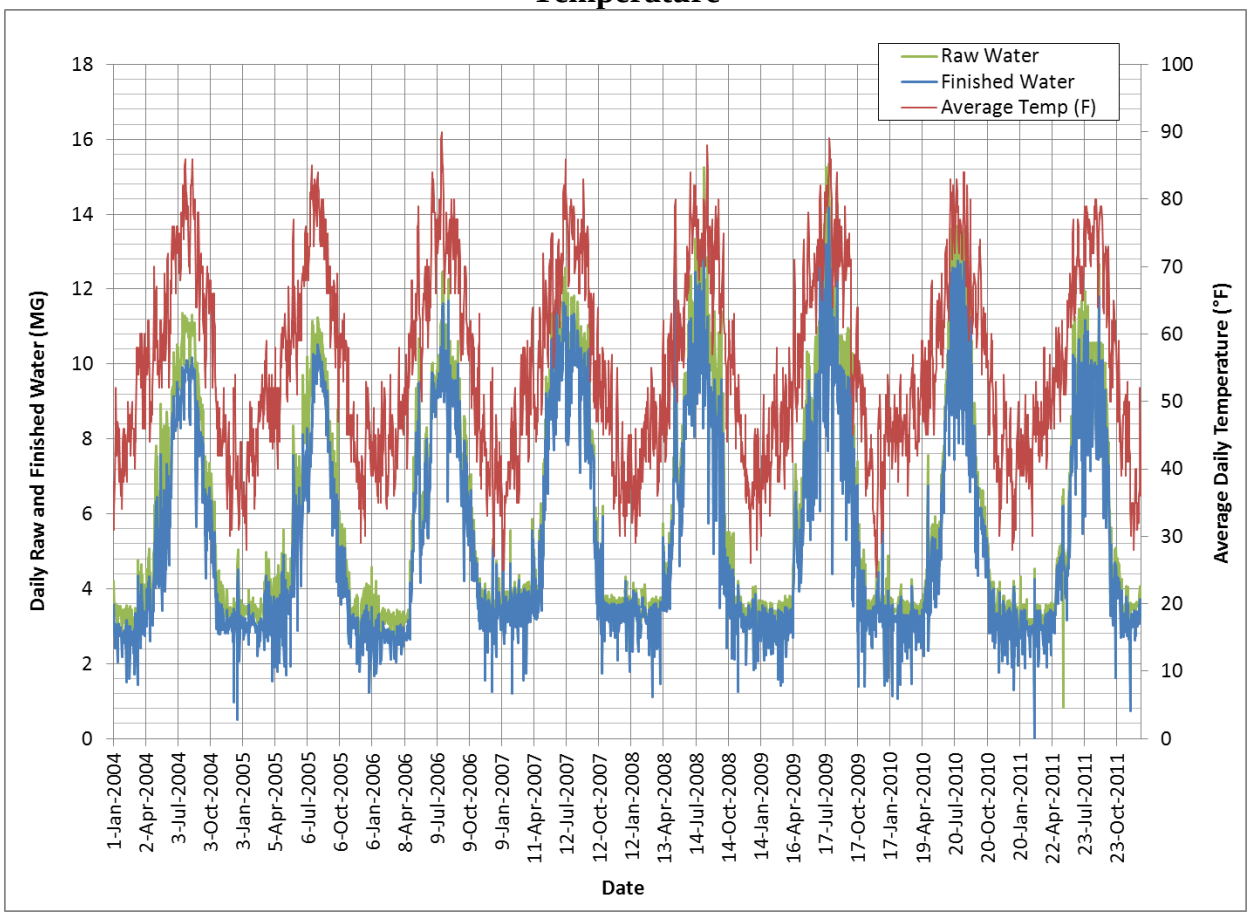
The 2004 WTPFP noted an increase of approximately 3 percent of recorded values for raw and finished water flow rates between 2001 and 2002, attributing the increase to installation of the new SCADA system. The plant staff was of the opinion that the old signal converters may have inadvertently dampened the flow signal, causing the measured flow rate to be as much as 10 percent less than the actual flow rate. There was an observed steady increase in annual average production as measured by the WTP effluent plant flow from 1999 to 2003. The trend of increased production has generally continued from 2004 to 2011.

Figure 2-1 presents the historic average daily raw water volume and finished water production from January 2004 to December 2011. Table 2-1 presents a summary of the plant production data including: annual average flow, average peak and off-season flow, minimum and maximum monthly average flows, maximum weekly average flows, and peak day flows. The City has been experiencing increasing water demands over the past decade. Average day production increased approximately 2 percent per year from 2004 to 2009. Demand in 2010 and 2011 decreased to 2004 levels, but this may have been due to mild summers and a depressed economy, and it is not anticipated that this trend will continue. The peak day production in 2012 of 13.6 mgd occurred on August 8. The summer of 2012 was drier and

warmer compared to the summers of 2010 and 2011. Figure 2-1 highlights that 2010 and 2011 did not experience the peaks in water temperature that prior years have experienced.

A maximum day production from the Grants Pass WTP of 14.2 mgd was observed on July 28, 2009. The highest average maximum monthly production of 10.5 mgd was observed in July 2010. Figure 2-2 displays the maximum daily operating rate of the plant from 2004 to 2011. Increasing demands can most likely be attributed to steady growth in the service area.

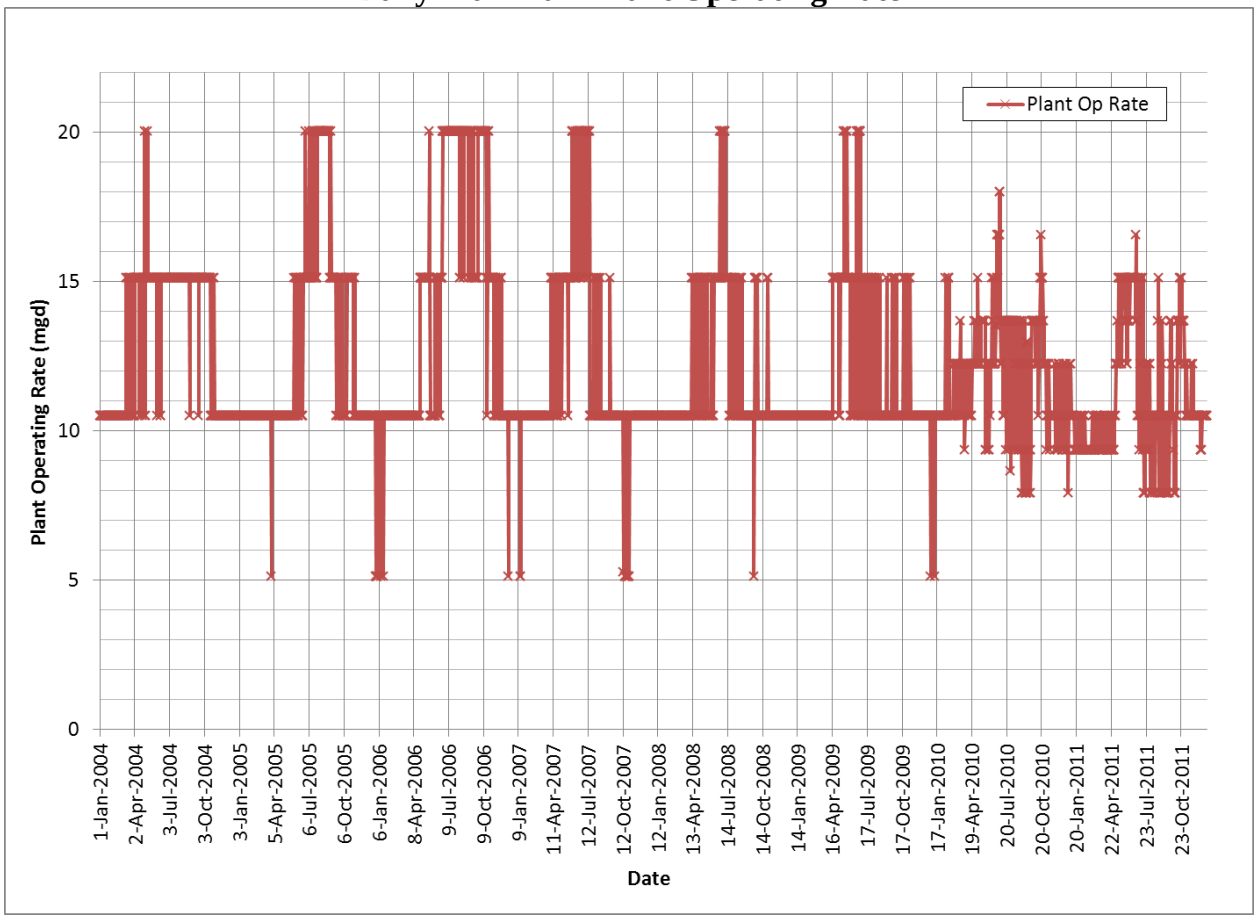
Figure 2-1
Historical Raw Water Intake and Finished Water Production and Average Daily Water Temperature



The flow data presented in Table 2-1 was used to develop peaking factors that are useful in water supply planning. The primary peaking factor is the ratio of peak day flow to annual average flow; this value ranged between 2.0 occurring in 2004 and 2007 to 2.5 occurring in 2009. Another important peaking factor is the ratio of peak month flow to annual average flow. For Grants Pass, this value ranged from 1.7 in 2007 to 2.0 in 2010. These values are consistent with those used for demand forecasting in the City's most-recent Water Distribution System Master Plan, where peaking factors of 2.2 and 1.8 were used for the peak day and peak month flows, respectively. This is in agreement with other recent studies on systems in the Pacific Northwest where maximum day peaking factors typically varied

from approximately 2.0 to 2.5. The peaking factors for the City system are consistent with these regional numbers.

Figure 2-2
Daily Maximum Plant Operating Rate



Typical Operations

The WTP operates in a daily start-and-stop mode for most of the year to minimize labor costs. The plant currently has 5 FTEs and uses seasonal employees when needed for a total of 6.0 to 6.5 FTEs on an annual basis. During the winter months, the plant is able to meet demands by typically running at 10.5 mgd for 8 hours per day. During the spring and fall, the plant historically ran at 10.5 mgd or 15.1 mgd for 8 to 12 hours per day. During the summer, the plant has run at the15.1 mgd or 20 mgd flow rate for 12 to 16 hours per day. The plant switched to 24 hour per day summer operations starting in 2007 at reduced flow rates. After the raw water pumps were equipped with VFDs in 2010, the plant has had more operational flexibility with respect to flow rates.

Table 2-1
Water Treatment Plant Production Summary[1]

Year		Flow (mgd)							
		2004	2005	2006	2007	2008	2009	2010	2011
Annual Average		5.0	4.8	5.3	5.8	5.5	5.6	5.2	5.0
Peak Season Average[2]		8.1	7.6	8.4	9.4	9.0	9.1	8.6	8.3
Off-Season Average[3]		3.4	3.3	3.7	4.0	3.8	3.8	3.4	3.3
Minimum Monthly Average	**Month**	Feb	Dec	Jan	Nov	Mar	Jan	Dec	Mar
	Flow	2.6	2.7	2.7	3.3	3.1	2.9	3.0	2.9
Maximum Monthly Average	**Month**	Jul	Aug	Jul	Jul	Jul	Jul	Jul	Aug
	Flow	9.3	9.6	9.6	10.1	10.	10.3	10.5	9.4
Maximum Weekly Average	**Week**	8/12-8/18	8/5-8/11	7/22-7/28	7/1-7/7	7/15-7/21	7/29-8/4	8/12-8/18	9/2-9/8
	Flow	9.9	10.0	10.2	11.1	10.8	11.2	11.5	9.5
Maximum Daily	**Date**	8/12	8/5	8/10	6/15	8/6	7/28	7/28	9/3
	Flow	10.2	10.5	11.7	11.9	13.9	14.2	12.8	11.8

Notes
1. Values as reported from plant effluent meter
2. Peak season average from June to September
3. Off-season average from January to May and October to December
4. From 1999 to 2003: Average day demand of 4.7 mgd, peak day demand of 10.5 mgd

Raw Water Quality

Five raw water quality parameters were analyzed as part of this review:

- turbidity,
- temperature,
- pH,
- alkalinity, and
- TOC.

These parameters are typically of most importance when evaluating a treatment plant's overall performance. A discussion of each of these parameters is presented as part of this section.

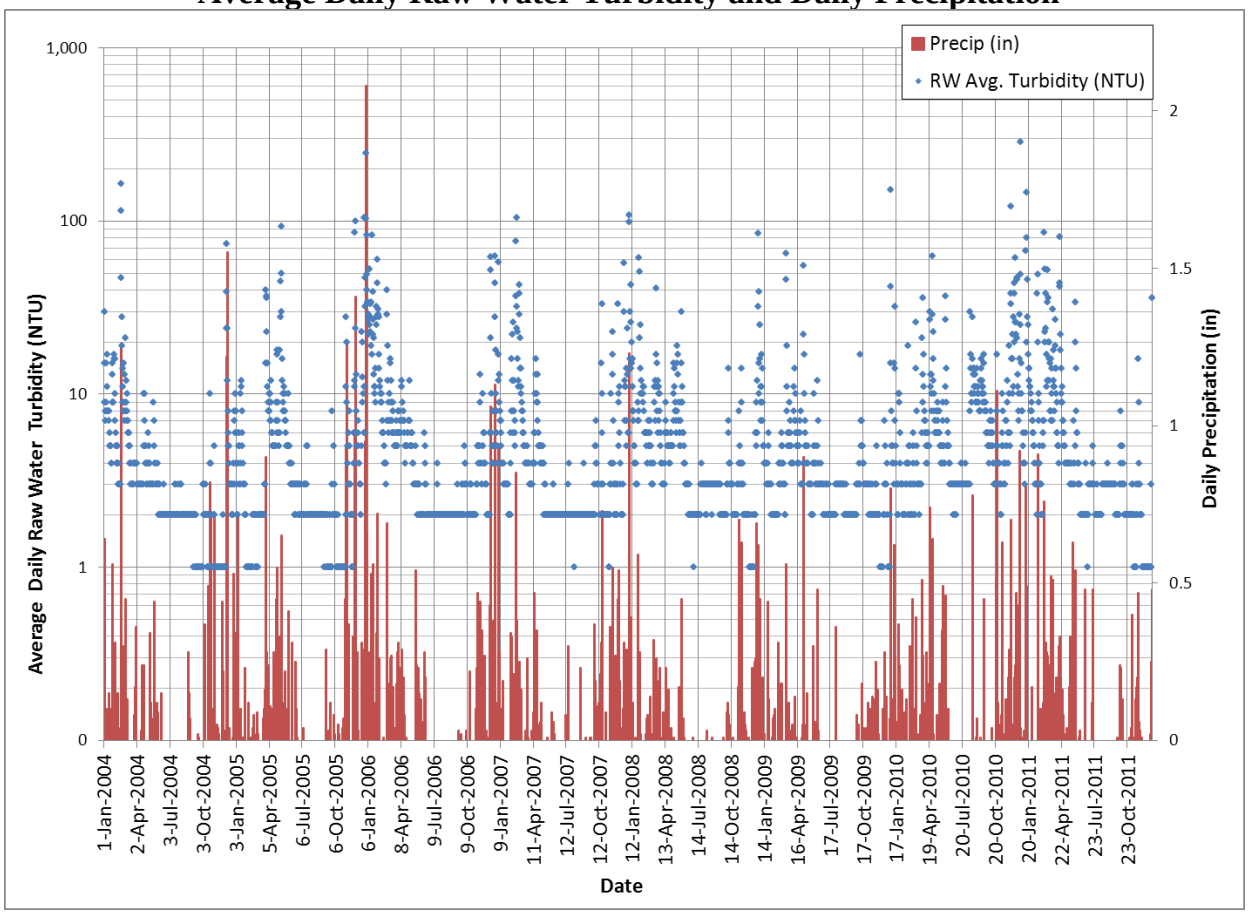
Turbidity

Turbidity is a measure of light penetration through a water sample and is indicative of the amount of particulate matter in the sample. It is measured in nephelometric turbidity units, or NTUs. Water with lower turbidity is typically easier to treat and usually requires lower chemical doses for optimum coagulation and filtration. High turbidity levels can reduce the

effectiveness of disinfection treatment processes and can provide a medium for the growth of microorganisms.

The raw water turbidity from the Rogue River has historically been low and moderately variable during the majority of the year. Increases in raw water turbidity generally correspond to high rainfall events. Figure 2-3 presents the average daily raw water turbidity, as well as the observed daily precipitation, between January 2004 and December 2011. The lowest turbidity periods occur during the warmer, drier months and the highest turbidity periods occur during the wet weather months.

Figure 2-3
Average Daily Raw Water Turbidity and Daily Precipitation



Average turbidities were generally less than 6 NTU from May to October; minimum turbidities were as low as 1.0 NTU during these months. Between November and April, turbidities typically averaged 9 NTU, with average maximums approaching 200 NTU. The highest average day raw water turbidity was reached in December 2010 when a daily average turbidity value of 286 NTU was observed in the raw water. A daily maximum turbidity spike of 787 NTU was observed in December 2005.

During the past few years, the WTP has experienced less predictable turbidity trends and the turbidity values have been more variable than they were in the past twenty years. This is attributed to the removal of the Gold Ray Dam, located on the Rogue River upstream of the WTP raw water intake, which began in the summer of 2010. Especially notable was the coffer dam failure in August 2010 that caused a breach in the dam. The plant experienced turbidity spikes of over 100 NTU; the August average turbidity is usually around 3 NTU. Since dam removal, the plant staff has noticed an increase of sediment accumulation on the base of the raw water intake screens, and has had to modify cleaning operations. The volume of solids that collect in the sedimentation basins has also increased, resulting in higher volumes of solids that needed to be handled and dewatered at the plant. The effects of the dam removal are not expected to be long-term, but it is currently unknown what effects the dam removal will have on turbidity and plant operations in the future.

Temperature

The temperature of raw water impacts water treatment by affecting the rate of chemical reactions, including disinfection and the formation of disinfection byproducts, floc formation and settling, and filter performance. As the temperature of the raw water increases, chemical doses generally decrease for floc formation, settling, filtration, and disinfection. An increase in optimal filter backwash rates results from an increase in water temperature due to the decreased viscosity of the warmer water.

Figure 2-4 shows that the maximum daily temperature of the raw water entering the WTP varies by season. From 2004 to 2011, winter temperatures averaged approximately 43.7 °F (6.5 °C) and summer temperatures averaged approximately 63.6 °F (17.6 °C). The lowest observed temperature in the time period was 33.3 °F (0.7 °C) on December 9, 2009. The highest observed temperature in the time period was 70.0 °F (21.1 °C), occurring on July 22, 23, and 24, 2004. Temperatures of 69.7 °F (20.9 °C) were also observed on July 24 and 25, 2010.

Raw Water pH

The acidic or basic nature of water is measured by pH and can be indicative of the water's corrosiveness. A pH of 7.0 represents neutral conditions, and pH values greater than 7.0 are generally considered less corrosive. Lower pH values usually indicate corrosiveness, which can lead to leaching of toxic metals into the water system and potential degradation of conveyance facilities. In water treatment, pH is also important because of its impacts on coagulation performance and chemical disinfection. A pH in the range of 6.0 to 7.0 is considered optimum for aluminum sulfate (alum) coagulation, and lower pH values are often desirable to enhance the removal of dissolved organic carbon. Lower pH values are often desirable for enhanced disinfection with chlorine. The formation of DBPs such as THMs and haloacetic acids is affected by the pH of the water during and following chlorination.

In plants lacking the ability to adjust pH at several points throughout the treatment process, corrosion control targets typically govern the pH, with perhaps some sacrifice in coagulation

and disinfection performance. The addition of certain water treatment chemicals alters the pH. Aluminum sulfate depresses the pH, while NaOCl increases the pH.

Figure 2-4
Maximum Daily Raw Water Temperature

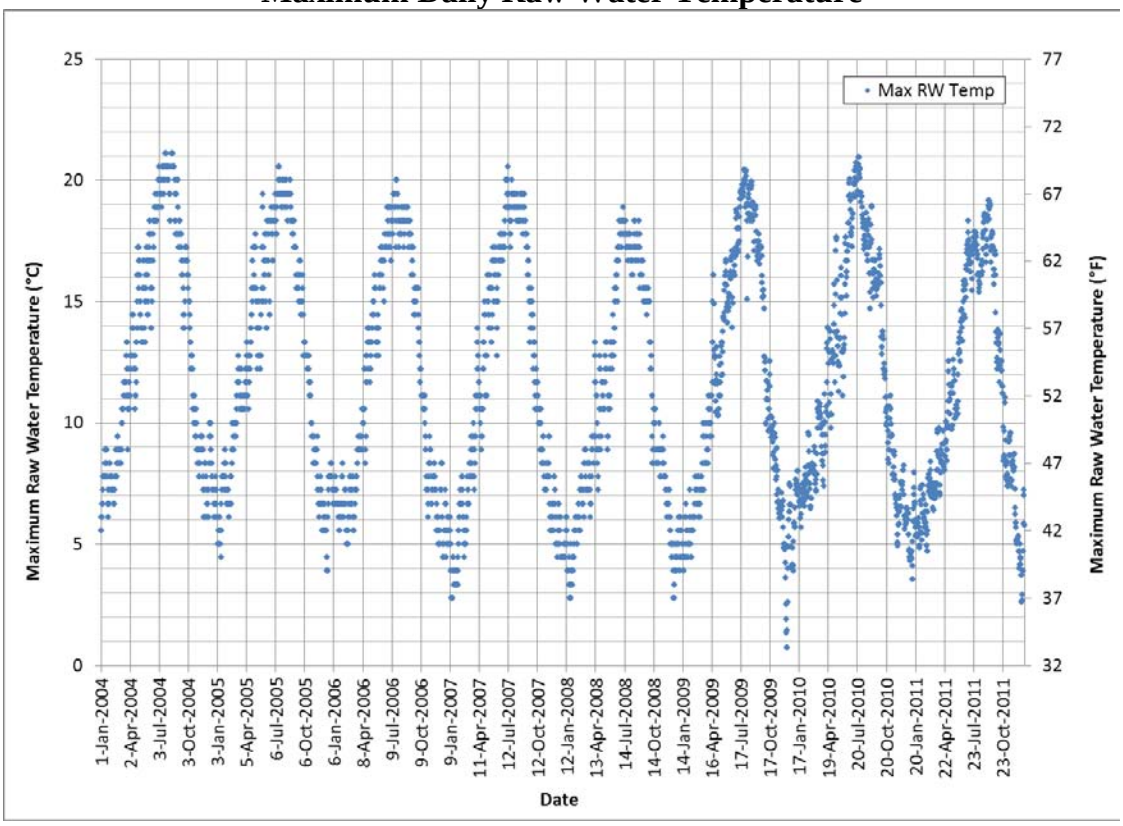


Figure 2-5 presents the historical raw water pH values recorded at the WTP between January 2004 and December 2011. As shown in Figure 2-5, the pH of the raw water from the river typically varies between 7.4 and 8.3 throughout the year, with average values between 7.6 and 8.0. Historically, pH peaks a few times each calendar year with the most pronounced peak occurring in mid-spring and a secondary peak occurring in the early fall, probably corresponding to algal activity in the river. Historic minimums occur in the winter months due to higher precipitation. The lowest observed raw water pH was 7.3 in December 2006. The highest observed pH was 8.6 in March 2005. Raw water pH can also be affected by algae throughout the summer, with diurnal variations between 7.5 and 8.5.

Figure 2-6 shows the historical finished water pH values recorded at the WTP between January 2004 and December 2011. Finished water pH has increased from 2007 to 2011 due to the reduction of alum usage and use of a new primary coagulant. The reduction in alum

Figure 2-5
Average Daily Raw Water pH

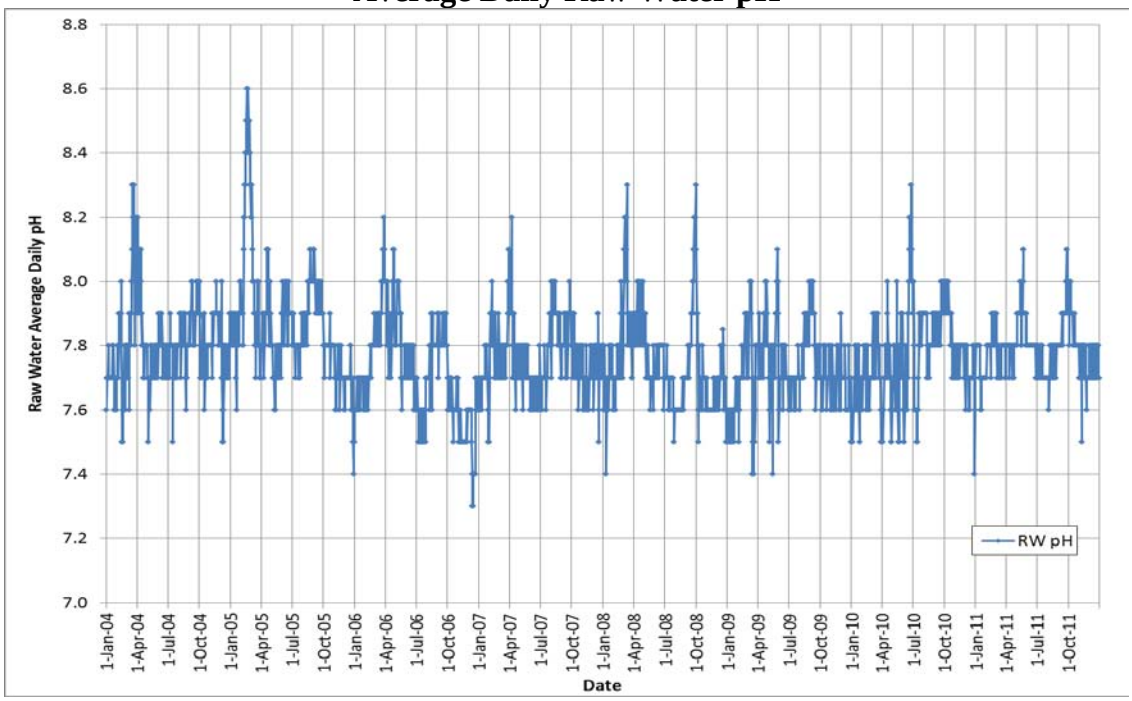
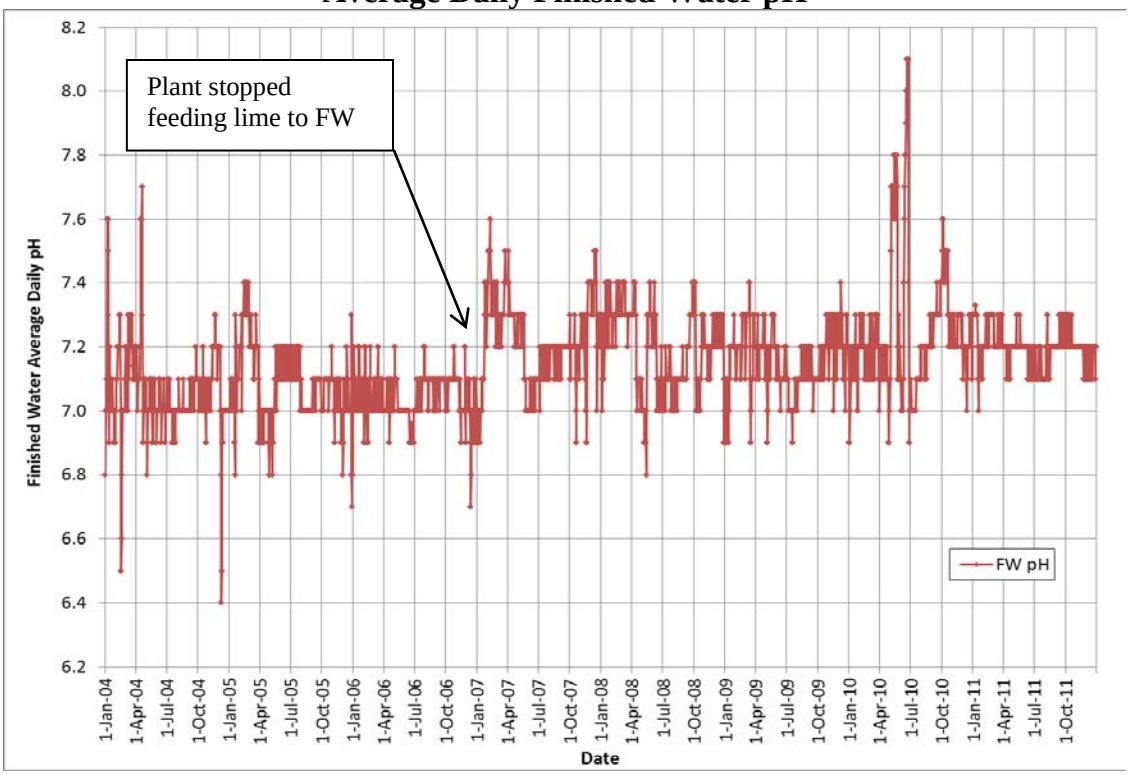


Figure 2-6
Average Daily Finished Water pH



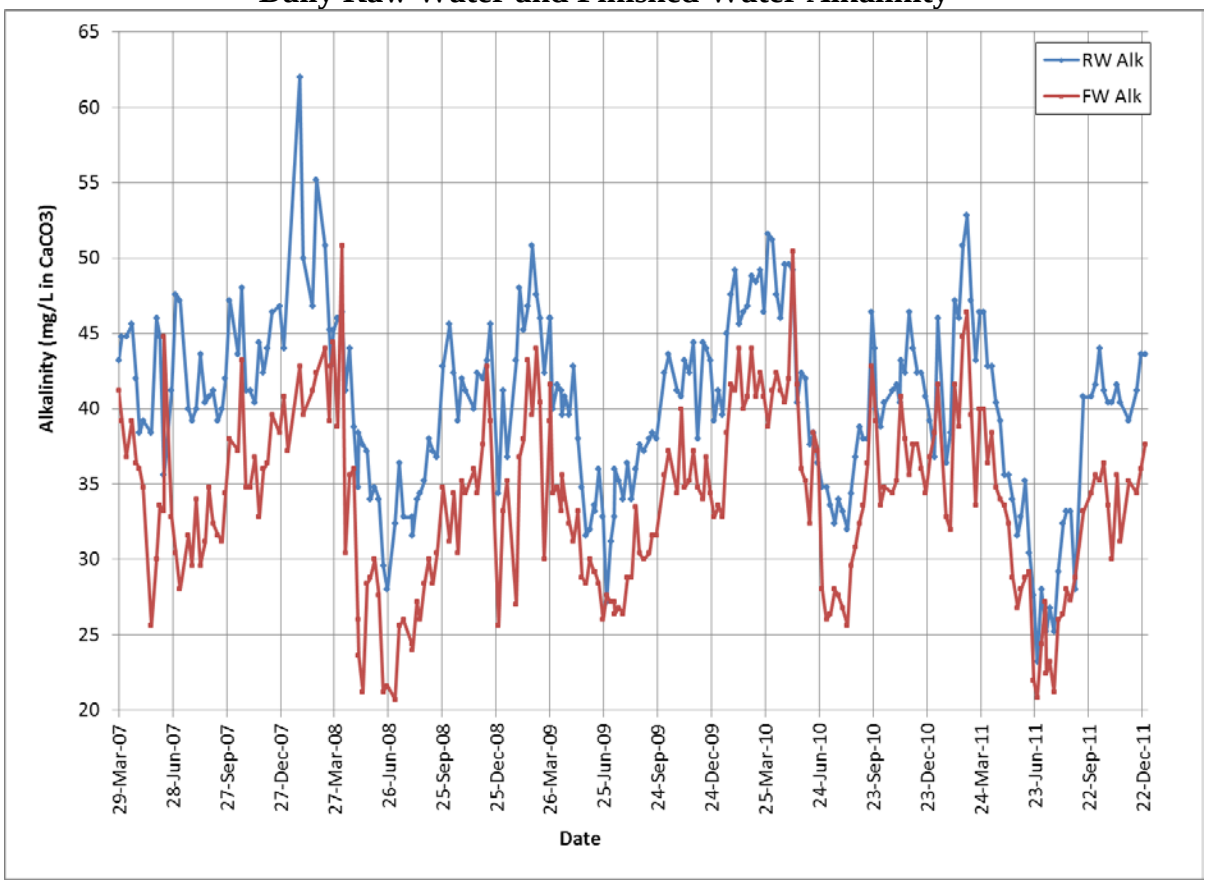
usage influenced the staff's decision to remove the lime system since post-filter pH adjustment is no longer practiced.

Alkalinity

Alkalinity is important in water treatment because of its impact on pH stability, coagulation performance, and corrosiveness. Alkalinity greater than 20 mg/L as $CaCO_3$ is generally considered adequate for aluminum sulfate coagulation and for improved pH stability in the distribution system. Alkalinity can also impact TOC removal requirements, depending on raw water organic concentrations.

The raw water alkalinity of Rogue River water varies seasonally as depicted in Figure 2-7, and seasonal trends seem to follow pH variability. The raw water alkalinity can be as low as 20 mg/L during winter periods and can be as high as 62 mg/L during the summer. When the alkalinity is low and turbidities are high, higher alum doses are required which can further depress the pH below optimum coagulation conditions. Using a coagulant which does not depress the pH or affect alkalinity during periods when the raw water turbidity is increased has eliminated the need to add an alkali to the raw water. Figure 2-7 also shows that finished water alkalinity is lower than the raw water alkalinity.

Figure 2-7
Daily Raw Water and Finished Water Alkalinity



Organic Content

The natural level of organic matter in the raw water can affect its treatability as well as other parameters, including chlorine demand, DBP formation, and tastes and odors. Organic content can be derived from the natural decay of plant life, as in humic and fulvic acids, or the presence of algae, or in some cases, from human activities. As the concentration of organic matter in the raw water increases, the need for chemicals such as alum and chlorine also typically increases. Since DBPs result from chlorine's reaction with organic matter, higher concentrations of organic matter in raw water usually result in higher levels of DBPs in the distribution system. Elevated algae concentrations can sometimes create difficult treatment conditions and can interfere with coagulation, cause filter clogging, or create nuisance tastes and odors, depending upon the type and concentration of the algae.

Total organic carbon is a general measure of the natural organic matter present in water. This parameter is sometimes used as an indicator of DBP formation potential. Total organic carbon is also important because existing regulations intended to minimize DBP formation require the removal of a fraction of the overall raw water TOC through the treatment process, depending on the raw water TOC concentration and alkalinity.

The Grants Pass WTP staff has been monitoring TOC concentrations in the raw and finished water at least monthly since 2002. Results from 2004 to 2010 are presented in Figure 2-8. The data suggest that the TOC concentrations in the raw water are comparable to other U.S. surface water supplies, typically ranging between 0.5 to 5 mg/L, and slightly higher than other similar Pacific Northwest surface water supplies, which often range between 1.0 to 3.0 mg/L. There were several samples prior to 2008 that were above 2.0 mg/L, the current "trigger" concentration for TOC removal requirements under existing regulations. Since 2008, there have only been four such instances. Further discussion of required TOC removal efficiencies and other regulatory issues associated with TOC are discussed in Chapter 3.

Because TOC analysis is expensive and labor-intensive, the 2004 WTPFP recommended the City consider purchasing a bench-top UV spectrophotometer and incorporating daily UV absorbance monitoring at the WTP as a surrogate measure for TOC. Dissolved and soluble organic carbon absorbs UV light at a wavelength of 254 nm. A spectrophotometer measures the percentage of UV absorbance, a value directly proportional to TOC concentration. Once calibrated, UV_{254} readings can be correlated to TOC concentrations. UV_{254} sampling is a relatively inexpensive and simple alternative to off-site lab analysis of TOC. The plant began recording UV_{254} in July 2004. For comparison, Figure 2-9 displays raw water TOC and UV_{254} readings.

Taste and Odor

The Grants Pass WTP does not typically experience significant taste and odor issues. Typically, WTPs that use the same source water have similar taste and odor characteristics. However, the taste and odor events related to algae that occur at the Medford Water Commission WTP upstream rarely occur at the Grants Pass WTP.

Figure 2-8
Raw Water and Finished Water Total Organic Carbon

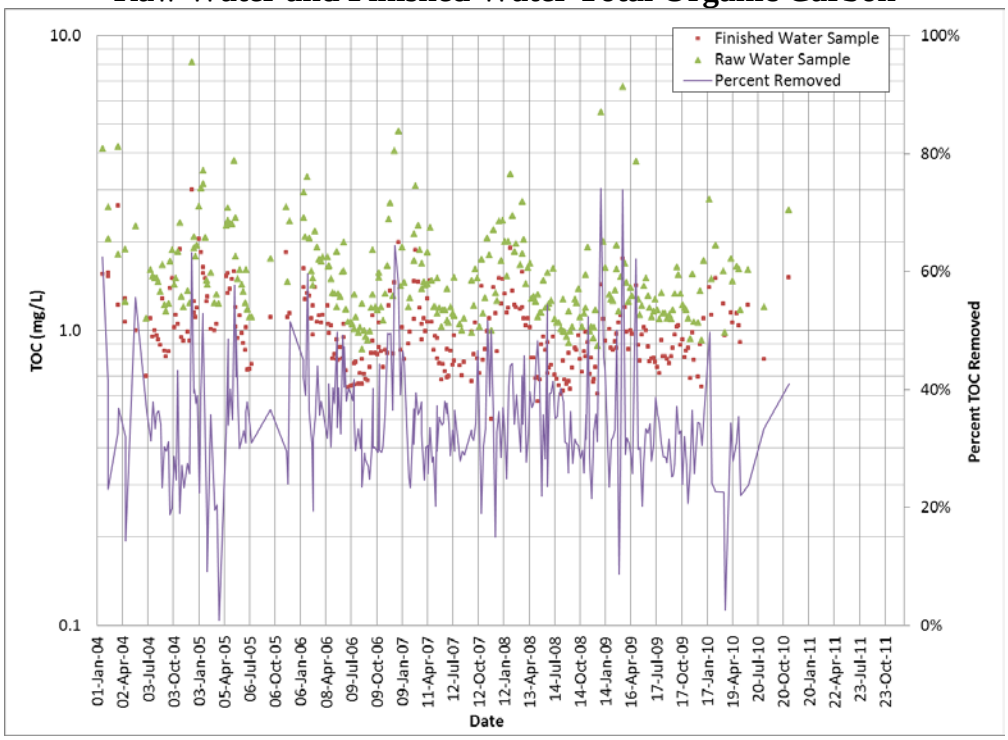
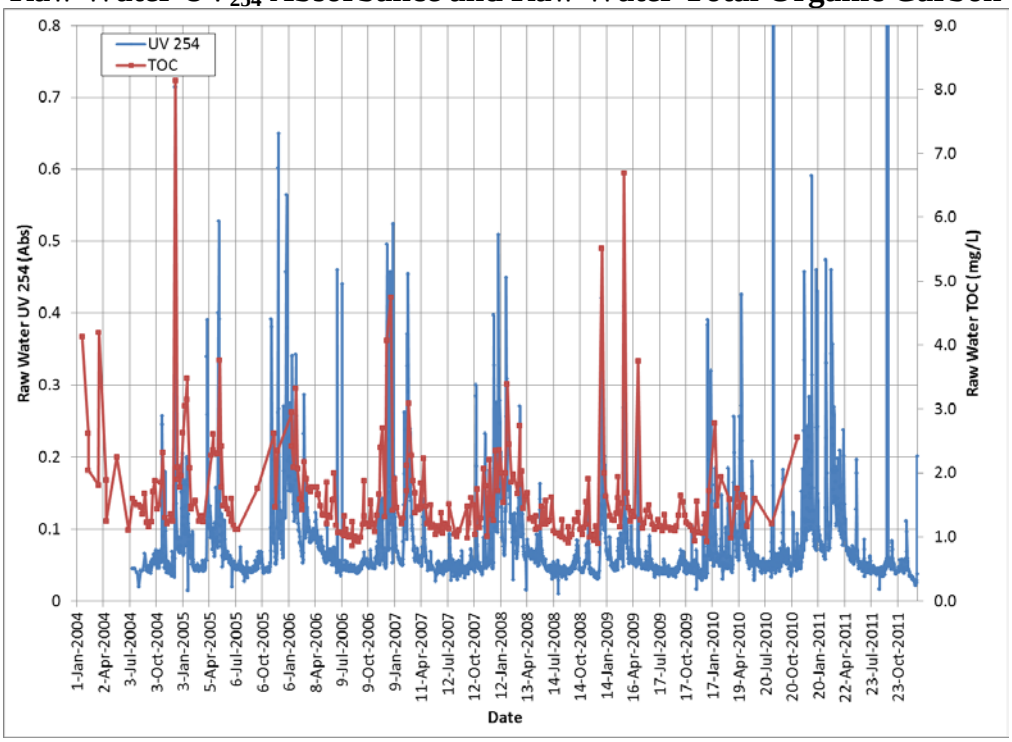


Figure 2-9
Raw Water UV_{254} Absorbance and Raw Water Total Organic Carbon



Chemical Usage

The four major chemicals currently used at the Grants Pass WTP are:

- alum
- ACH,
- filter aid polymer, and
- liquid sodium hypochlorite.

Liquid alum and ACH are used as the coagulants and are fed year-round. The polymer is used to condition the water entering the filters for improved filter performance. Sodium hypochlorite is added to the raw water and finished water as a disinfectant. Hydrated lime and potassium permanganate are chemicals that were used in the past. As noted above, lime use has been discontinued, and potassium permanganate is used infrequently. A brief discussion of each chemical is presented as part of this section.

Aluminum Sulfate

Liquid alum is stored as a 50 percent solution, by weight, and fed via metering pump to the raw water pipeline upstream of the static mixer prior to the flow split to the basins. The addition of alum to the raw water destabilizes negatively charged suspended particles, thereby allowing the formation of insoluble floc particles via coagulation and flocculation, and their subsequent removal via clarification and filtration.

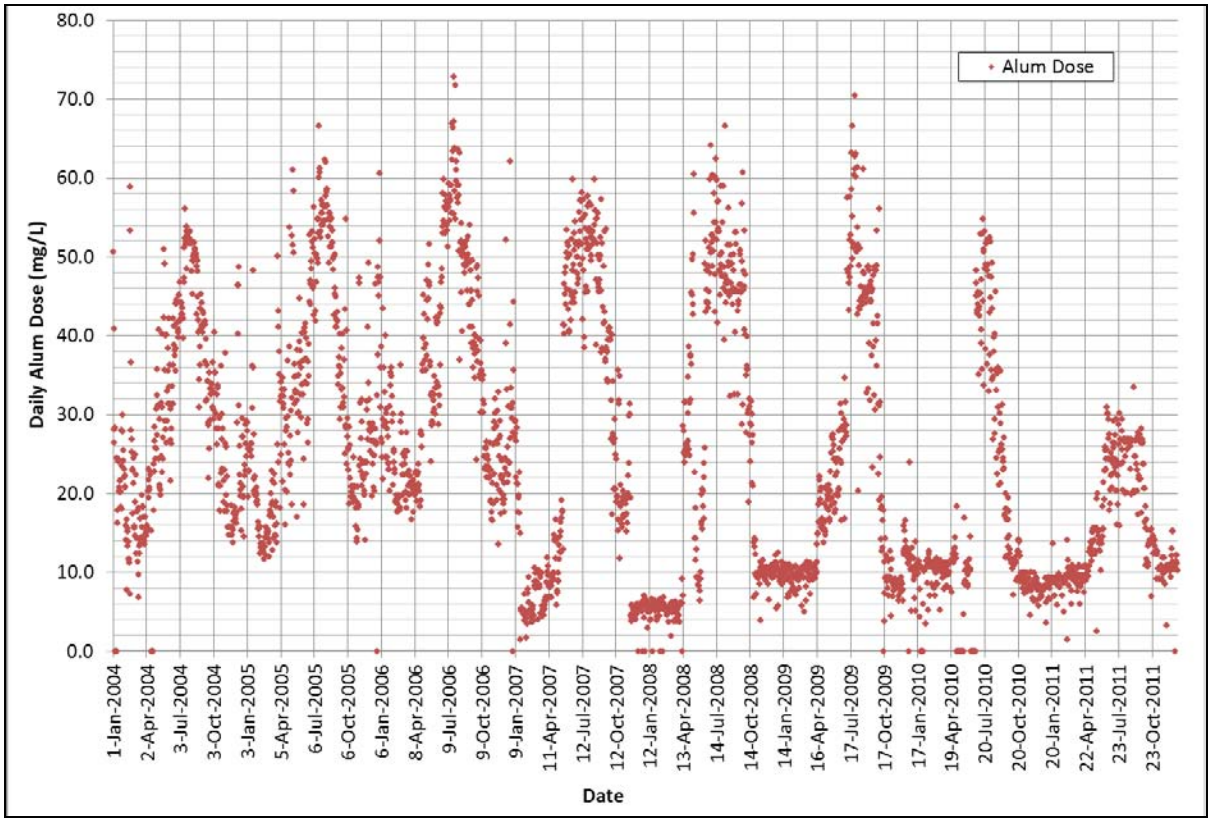
Figure 2-10 shows the annual trends in alum usage between January 2004 and December 2011. The required alum dose varies throughout the year. From 2004 to 2006, when alum was the sole coagulant used, the typical off-peak season alum doses averaged 27 mg/L as dry alum while peak season alum doses averaged 21 to 22 mg/L as dry alum. The highest alum doses have typically been above 60 mg/L as dry alum during fall and winter because of high turbidity events. The plant used an average of 200 tons of alum per year from 2004 to 2006. In response to the 2004 WTPFP, the plant staff began experimenting with different coagulants (ACH and PACl) and started using these other coagulants with alum intermittently from 2007 to 2008. Starting in the fall of 2009, the plant began feeding alum and ACH concurrently. In all cases, use of an additional coagulant has been able to reduce alum dosages and multiple benefits have been observed including less pH depression and lower solids production rates. In addition, the plant was able to stop feeding lime for pH adjustment. Filter performance may also be enhanced by the current coagulation process as the floc formed is generally stronger and has a higher shear resistance within filter media. Alum usage from 2010 and 2011 averaged 93 tons per year.

Other Coagulants

ACH and PACl are generic terms used to describe different formulations of proprietary coagulants that are derivatives of the general base molecular formula of an aluminum

chlorohydrate molecule. These proprietary formulations vary in strength, pH, basicity, freezing point, and specific gravity. ACH and PACl offer many benefits to optimizing coagulation strategies. They do not depress pH like alum and, as a result, reduce the need of an alkali addition to adjust the pH.

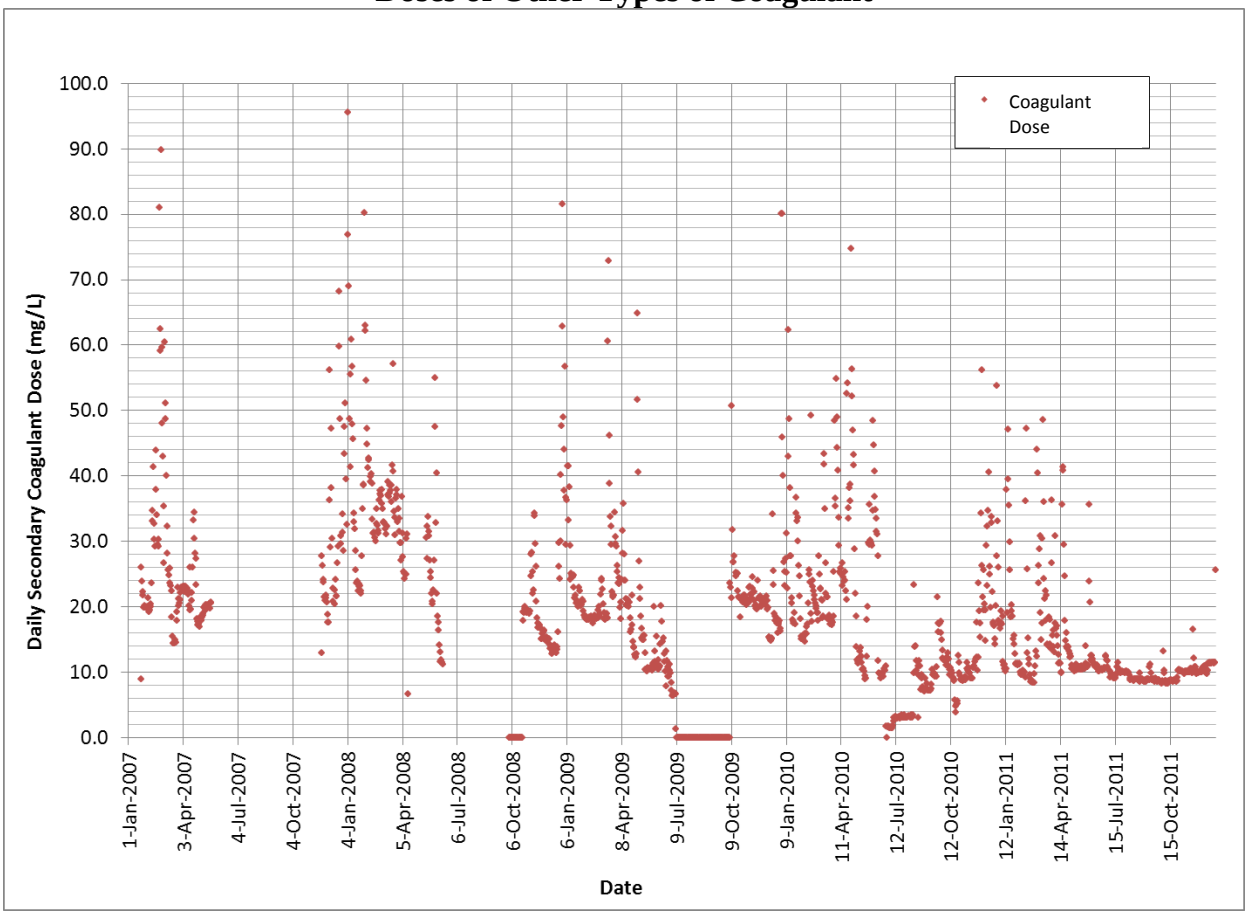
Figure 2-10
Alum Doses



From 2007 to 2008, the plant tested Pass-C, a PACl derivative, in conjunction with alum during different seasons. Except for one week in January 2010 when the plant tested NIAD I-5, another PACl derivative, as the sole coagulant, the plant began feeding an additional coagulant with alum during daily operations in the fall of 2008. In May 2010, the plant switched from Pass-C to T-Floc B-135, a derivative of ACH. Since that time, the plant has transitioned from using alum as its primary coagulant to ACH as its primary coagulant. Alum is now used as a supplementary coagulant.

Use of ACH as a primary coagulant has reduced alum usage at the City's WTP. Because of this, a tank which was formerly used to store alum is now used to store ACH in bulk. A separate metering pump doses ACH to the injection location. Figure 2-11 shows doses of coagulants other than alum from 2007 to 2011.

Figure 2-11
Doses of Other Types of Coagulant



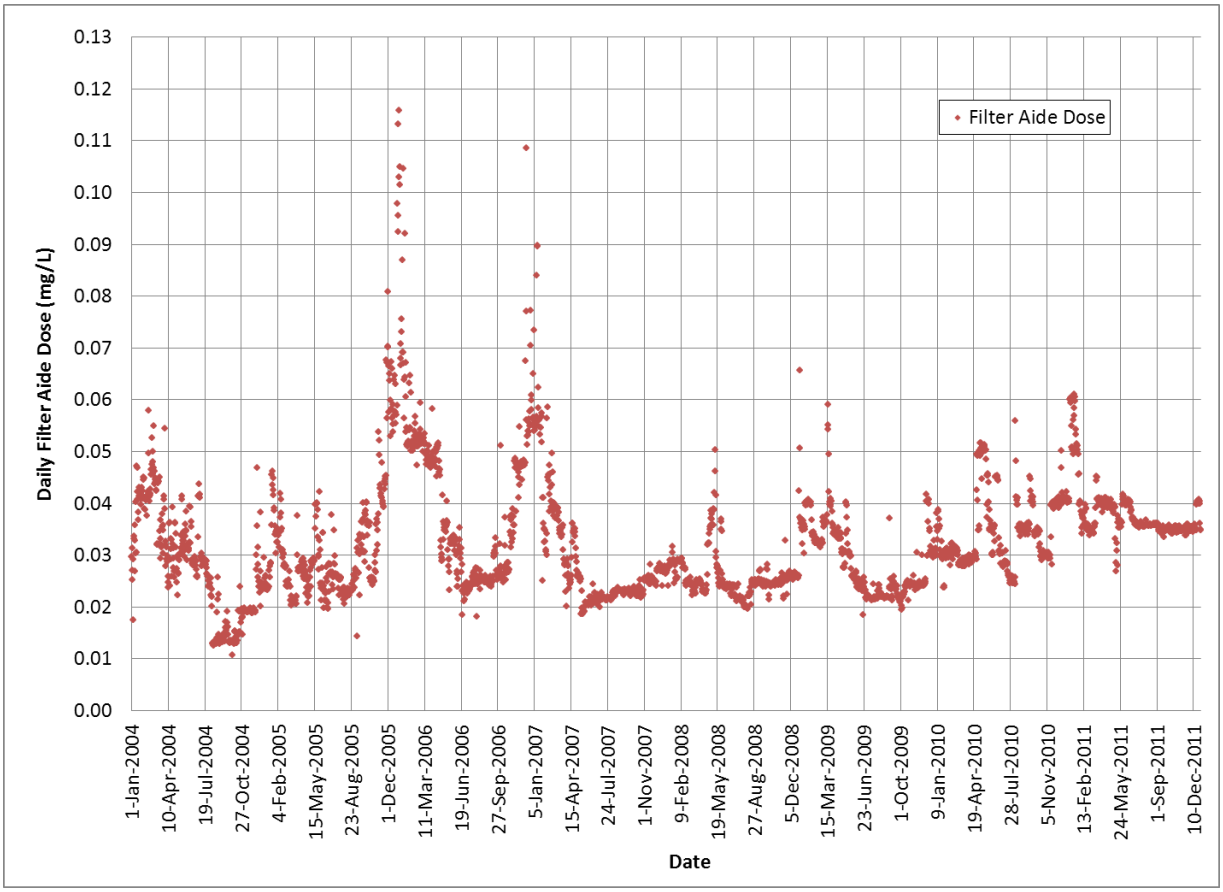
Polymer Filter Aid

The Grants Pass WTP currently uses a low-molecular-weight polymer as a filter aid. The dry polymer is mixed and aged with water, then fed via metering pump and carrier water to the filter influent. Flows are split eight ways to each filter using rotameters. Filter aid polymer is used continuously throughout the year and total daily usage is monitored and recorded. The polymer's role in improving overall turbidity removal at the Grants Pass WTP is important. When introduced to the settled water, the polymer helps make the alum floc that leaves the sedimentation basins "stickier." This property helps the filters retain the floc better and minimizes turbidity "breakthrough." If the filter aid were not added, the filtered water turbidity would be higher and filter run lengths would be significantly shorter due to premature breakthrough. This would require more frequent backwashing.

Figure 2-12 presents the historic average daily filter aid polymer dosages from 2004 to 2011. Filter aid polymer dosages tend to increase in the winter when water temperatures are low and decrease in the summer and early fall when the water is warmer. The average daily polymer dose was 0.027 mg/L during the summer, increasing to approximately 0.040 mg/L

in the winter and as high as 0.12 mg/L during winter's most challenging raw water conditions.

Figure 2-12
Filter Aid Doses



Lime

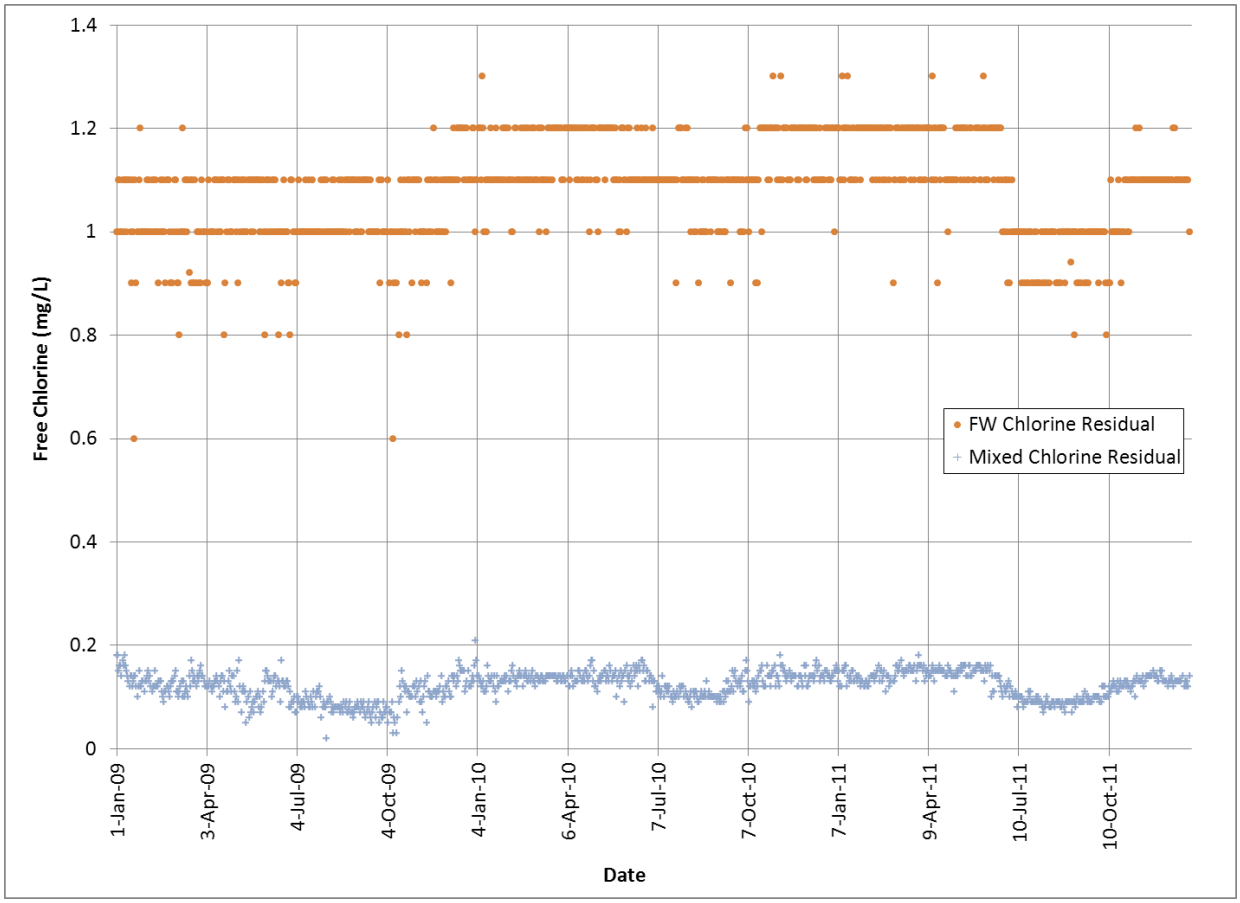
Lime was historically used to raise the pH by restoring alkalinity consumed through the coagulation process when alum was the sole coagulant. Plant staff maintains a target finished water pH of 7.2 for corrosion control. Since the plant has changed the coagulation approach and now uses ACH in addition to alum, the alum dose has decreased. The pH depression caused by alum has been to the point where lime is no longer needed to maintain proper finished water pH for corrosion control.

Sodium Hypochlorite

Hypochlorite is added to the raw water to assist in coagulation, control biological growth through the sedimentation basins, and for disinfection purposes. The target chlorine residual exiting the sedimentation basin is approximately 0.2 mg/L to ensure a measurable residual is maintained throughout the basins and to ensure disinfection compliance. The plant has reduced the pre-chlorination dose over the past few years to minimize DBP formation.

Chlorine addition to the finished water is intended for disinfection purposes and is added to maintain a chlorine residual in the distribution system. Chlorine is "boosted" throughout the distribution system (up to three times for some parts of the system) for residual maintenance. Figure 2-13 shows the free-chlorine residual in the treated raw water following chemical addition and rapid mixing by the 36-inch diameter static mixer. Pre-chlorination doses have typically ranged from 0.2 mg/L to 1.4 mg/L, although this range represents changes in operational strategy as well as fluctuations caused by normal operation. The figure also shows the free chlorine residual in the finished water effluent following post-chlorination. Finished water chlorine residuals are generally maintained between 0.9 mg/L and 1.3 mg/L with an average of approximately 1.1 mg/L.

Figure 2-13
Mixed and Finished Water Free Chlorine Residuals



Liquid sodium hypochlorite is stored at 12.5 percent solution in three 2,300-gallon fiberglass tanks located on-site. The hypochlorite system was installed in 2001 to replace the original gas chlorine injection system.

Additional Chemicals

In addition to the primary treatment chemicals used daily at the Grants Pass WTP, the plant also has the ability to dose $KMnO_4$ for taste and odor control. The use of potassium permanganate is rare. It was last used over a four-day period in December 2010 to January 2011 in response to a taste and odor event. Originally, the plant was designed to use PAC as an additional taste and odor control process, but PAC was rarely, if ever, used. The PAC slurry tank was converted to a solids mixing and conditioning tank and PAC can no longer be fed.

The WTP uses other miscellaneous chemicals for operational purposes. A long-chain polymer is applied to sedimentation basin residual solids during cleaning activities to aid in dewatering. Calcium thiosulfate is used to dechlorinate filter maintenance water and is also dosed to water for the intake structure wash system.

Plant Performance Data

The WTP staff keeps daily records of plant performance data that were used to assist in the evaluation of overall plant performance. This section summarizes the historic operating performance of the treatment processes including the sedimentation basins and filters.

Coagulation

The Rogue River water quality presents some treatment challenges at the WTP resulting from seasonal and diurnal variations in pH, seasonally variable turbidity, and temperature, as well as occasional taste and odor events. Excepting taste and odor, this variable raw water quality can significantly impact coagulation performance at the plant. Historically, these challenges were met by using a relatively high dosage of alum. This strategy resulted in high solids production and depressed pH which corresponded to an increase in pH adjustment chemical usage and cost and decreased overall plant efficiencies. The 2004 WTP Facility Plan suggested the use of a different coagulant to offset these deficiencies. Now that the plant is using ACH as a primary coagulant, overall alum usage has decreased by half. As a result, the plant operates at higher efficiencies and the use of a pH adjustment chemical is no longer necessary.

Clarification

The City's WTP relies on three sedimentation basins for clarification prior to filtration; no formal flocculation is provided in the basins. Basin 1 was constructed as part of the original plant; basins 2 and 3 were incorporated into the plant during the various plant expansions. The design of the basins are different and effluent water quality differs between the basins as a result.

The basins are each drained and cleaned at least twice per year. Prior to 2007, cleaning was restricted to off-peak seasons, as the plant required the full capacity to meet summer

demands. Now that the plant is operating 24 hours per day during the summer, each basin operates at a lower flow rate and it is possible to take a basin out of service for cleaning while still meeting peak season demands. As solids accumulate in the basins, the detention time decreases, which reduces the solids removal and disinfection performance of the basins.

The State currently rates the plant as "complete conventional," but the lack of formal flocculation and higher-than-desired surface loading rates of the basins could result in a future de-rating to a direct filtration plant. This would present significant challenges to providing disinfection during periods of high demands.

Typical Operations and Flow Control

Raw water flow is split into two pipes downstream of the static mixer; the first pipe leads to a slow mix basin for basins 1 and 2, the second leads to basin 3. Each pipe has a butterfly valve for flow control. A butterfly valve located at the influent to the slow mix basin can be used to control flow, but it is normally left open. The pipes and valves were designed to split the plant flow proportionally to each basin based on the basin's settling area. The proportions of flow reaching each basin are approximately 36 percent, 24 percent, and 40 percent of plant flow to basin 1, 2, and 3, respectively. Short-circuiting has caused flows through basin 3 to be reduced. The valves controlling the flows to each basin were set based on a plant flow of 20 mgd and the percentage of flow to each basin varies at lower plant flow rates unless the valves are manually adjusted.

Clarified water flows from the sedimentation basins to the filter influent channel. In general, filters 1, 2, and 3 are fed by basin 1; filters 4 and 5 are fed by basin 2; and filters 6, 7, and 8 are fed by basin 3. The clarified water trough is continuous between the filters and is intended to distribute the water evenly to the filters associated with each sedimentation basin. Because basin 3 is farther from basins 1 and 2 and has a longer pipe connection, the amount of water mixing and sharing between basins 1 and 2 and basin 3 may be somewhat restricted.

Sedimentation Basin Geometry

An optimal sedimentation basin is rectangular with a minimum length-to-width ratio of 4:1, a minimum length-to-depth ratio of 1:15 and a sufficient volume to keep mean flow velocity under 3.5 ft/min. Optimal basins provide approximately 20 to 30 minutes of flocculation and 90 to 120 minutes of sedimentation, or a total of 120 to 150 minutes of detention time. Baffles are useful to ensure good flow distribution and prevent short-circuiting. None of the three basins meet these optimal parameters.

Basins 1 and 2 are rectangular basins. Water enters at the south ends of the basins. Laminar flow conditions are improved in basin 1 by two baffle walls: one at the inlet, the second approximately half way along the length of the basins. Basin effluent collects in launders located on the north ends of the basins.

Basin 3 is the newest basin in the plant, built in 1983. Water enters this square basin via a central vertical pipe that discharges through ports located from 3 to 5.5 ft below the water surface. The water then flows under a circular baffle that extends from just above the water surface to 8 ft below. Water exits from the basin into one continuous square launder. Water from this square launder collects in a common trough that flows to the filter influent trough. Because its square shape and radial flow, basin 3 is vulnerable to short-circuiting. Despite the large volume of the tank, the path length from the inlet to the outlet is relatively short.

Based on these criteria, it is expected that basins 1 and 2 will be more efficient with solids removal than basin 3. Stable flow is difficult to maintain in basin 3 because its cross-sectional area is large in comparison to the cross-sectional area of flow. There are no automated solids removal mechanisms installed inside any of the basins, although provisions for future upgrades were included in the design of basin 3.

Sedimentation Basin Performance

Overall, the sedimentation basins provide satisfactory water for filtration during most of the year, as evident by filtered water turbidities. All basins experience challenges with regard to short-circuiting, high solids loading resulting from relatively high coagulant dosages, sub-optimal flocculation, and seasonal turbidity spikes. The basins are not equipped with any type of automated solids removal system. As solids accumulate in the basin, the effective volume of the basin is reduced which compromises flow characteristics and overall performance in the basin until the solids are removed. Without having continuous residual solids removal in the basins, basin cleaning events create large, “slug” doses of solids that present operational challenges. Basin 3 is especially vulnerable to short-circuiting or not clarifying as efficiently as basin 1 and basin 2, as indicated by filters 6, 7, and 8 needing more frequent backwashing. Plant staff observations and operating data support that the filters fed by basin 3 are backwashed approximately 25 percent more often than the rest of the filters.

Filtration

The plant has eight dual-media gravity filters of varying sizes and shapes, depending on the time of construction. Filters 1, 2, and 3, also called the East Filters, were constructed in 1931 as part of the original construction. filters 4 and 5, called the West Filters, were constructed as part of the 1950 plant expansion. Filters 6, 7, and 8, were added as part of the 1983 expansion project.

All of the filters which were constructed at the same time have the same individual surface areas, but the surface areas of filters in other groups are different. It is uncommon for a WTP to have variable filter shapes because demands on the filter support systems common to all filters (i.e. backwash pump, surface wash pump, washwater conveyance system, etc.) will vary according to the different filter surface areas.

The original filter design used mixed media with gravel support. Based on recommendations made in the 2004 WTPFP, the filters were modified in 2005 to use a deeper dual media with new underdrains that do not use gravel support. The current dual media design includes 20 inches of 1.0-mm anthracite over 10-inches of 0.5-mm sand. This new dual media has resulted in longer filter run times between backwashes and has improved overall plant production efficiency while continuing to produce low filtered water turbidities.

Typical Operations

The filters are operated by rate-of-flow control. Butterfly valves on individual filter effluent pipes modulate to maintain a specific filtration rate. Overall filter flow is adjusted to maintain a constant water level in the filter influent channel. Filter aid is dosed at the influent to each filter. The filters share common backwash pumps equipped with VFDs to provide variable flow rates depending on filter size and water temperature. Until an additional backwash pump was installed in 2012, there was no back-up supply for backwash water.

Turbidity

Each filter at the Grants Pass WTP is equipped with a turbidimeter to measure the turbidity of the individual filter effluent. Another turbidimeter is located in the filter gallery to measure the plant's combined filter effluent (CFE) turbidity. Data from each of these instruments is used for regulatory reporting. Figure 2-14 presents a summary of daily maximum combined filtered water turbidities between 2004 and 2011, taken from the plant's regulatory summary sheets reported monthly to the OHA. As shown in the figure, the maximum daily turbidity has always been less than 0.70 NTU, and is usually less than 0.10 NTU. Figure 2-15 presents a statistical summary of maximum daily combined filter effluent turbidities between 2004 and 2011 . From the figure, the plant has produced water with a turbidity of 0.05 NTU or less 95 percent of the time. The plant has normally performed well with respect to meeting the desired turbidity goal for optimal particulate removal.

Individual filtered water turbidities have also been recorded since 2004. These measurements are used to monitor filter performance and help decide when a filter needs to be backwashed. They are also used to determine when a filter-to-waste cycle should be stopped following a backwash.

All eight filters have produced filtered water turbidities under 0.15 NTU for at least 95 percent of the time. In general, all filters are performing well with regard to overall particulate removal.

Figure 2-14
CFE Turbidity Values

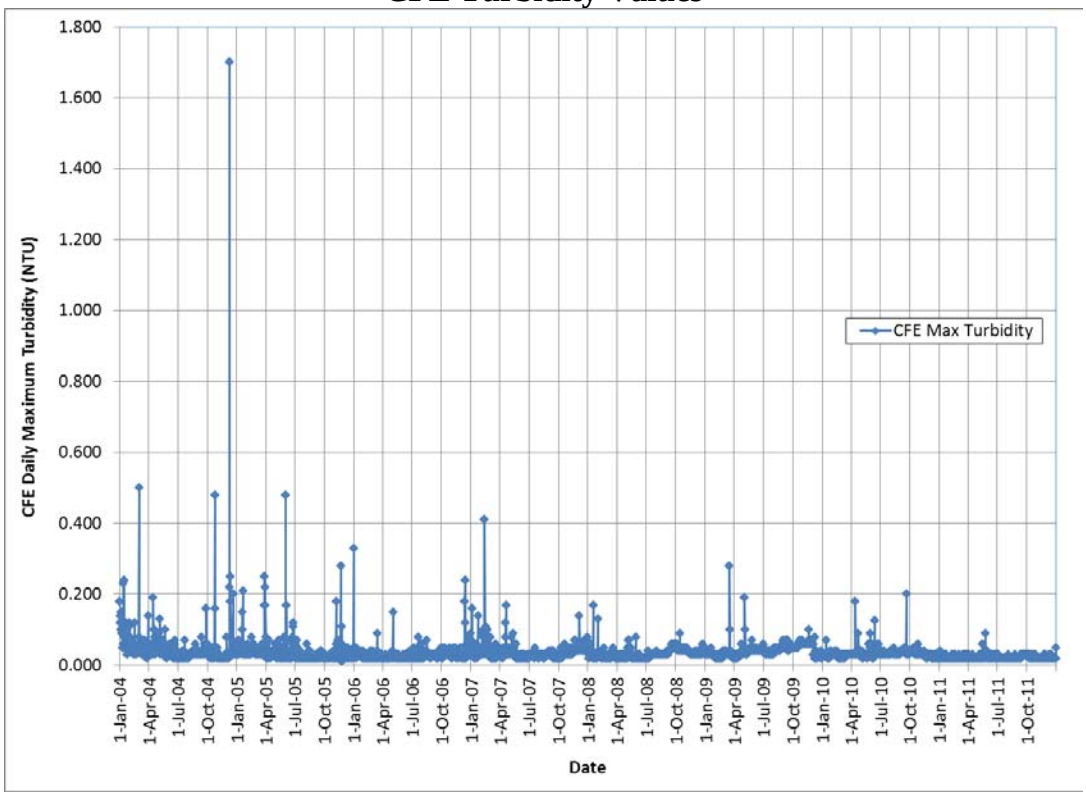
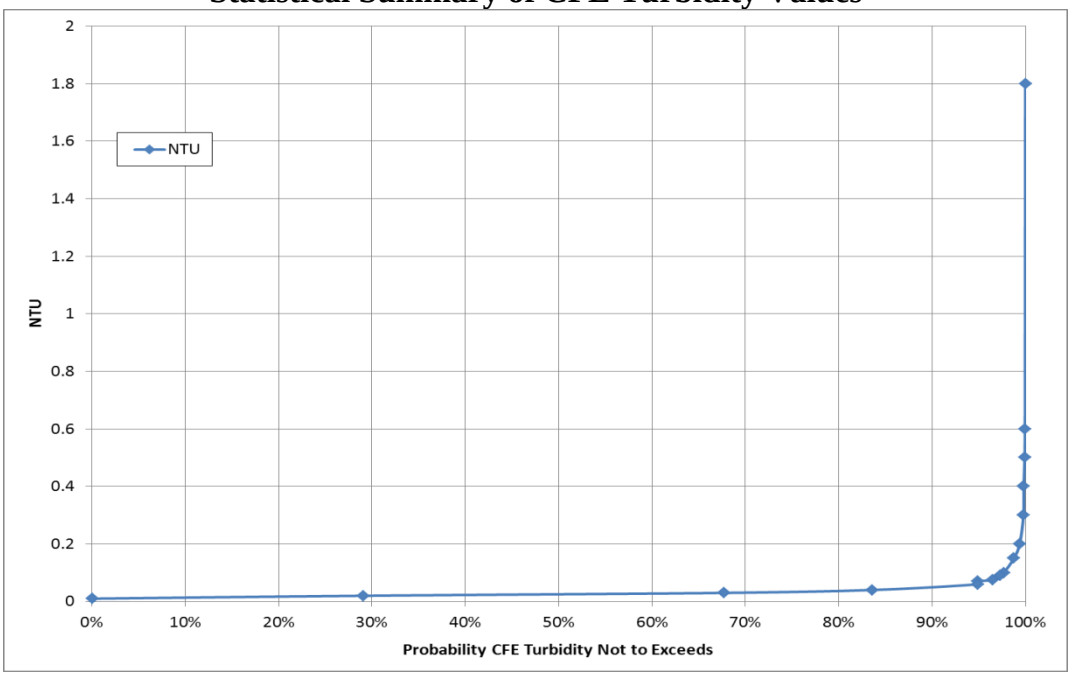


Figure 2-15
Statistical Summary of CFE Turbidity Values



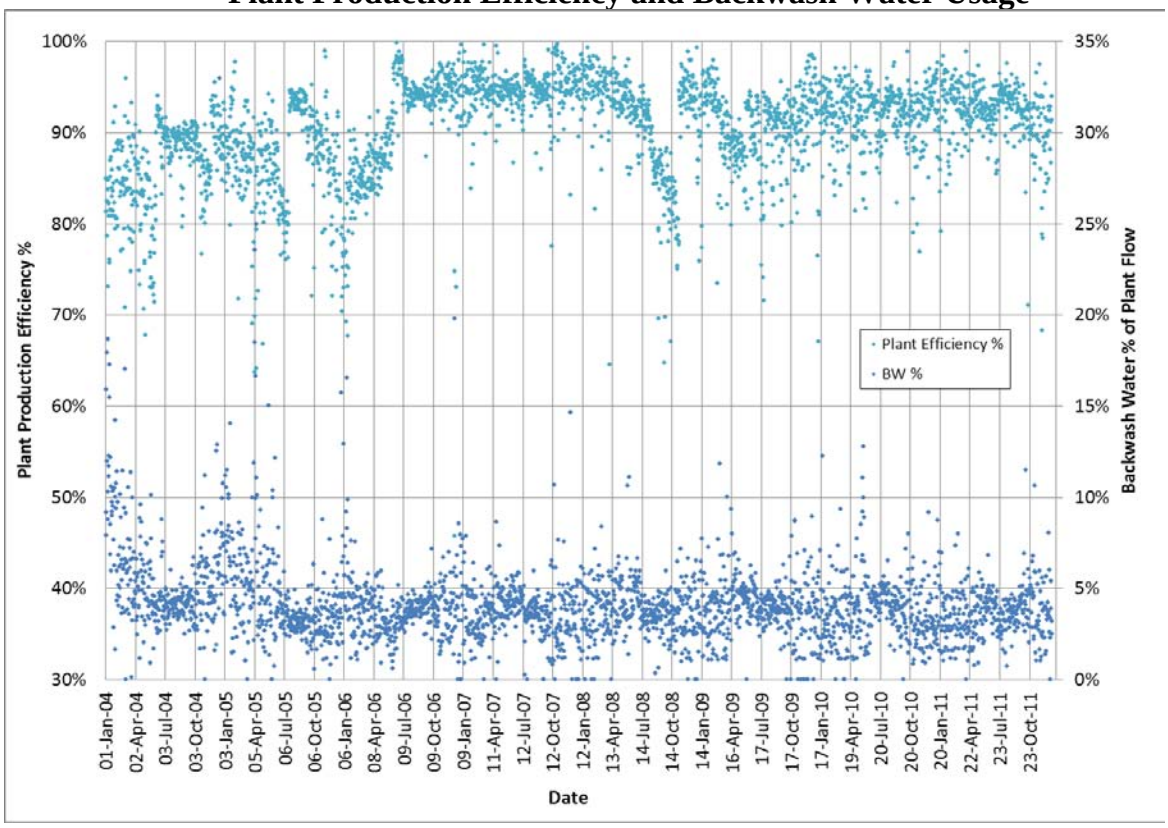
Filter Production Efficiencies

For a new surface water treatment plant, a typical suggested minimum overall efficiency is 97 percent. The City's WTP efficiency does not consistently meet this goal. The 2004 WTPFP identified a number of improvements that could be made to the filtration process to improve production efficiencies. As a result, the City has made the following upgrades:

- Filter media replacement and optimization
- Filter underdrain improvements
- Optimized filter backwash procedure and Unit Filter Run Volume optimization
- Addition of a second backwash pump to help provide backwash operational reliability

Figure 2-16 shows how plant efficiency has increased since 2004 and that the amount of backwash water used as a percent of plant flow has decreased. Prior to 2006, plant production efficiencies were in the range of 80 to 90 percent, while from 2006 onward plant efficiencies are observed to be consistently 85 to 95 percent. If basin 3 turbidities could be reduced, filters 6, 7, and 8 may require less frequent backwashing, resulting in increased plant efficiency.

Figure 2-16
Plant Production Efficiency and Backwash Water Usage



Operations and Maintenance

Historical operations and maintenance costs for the WTP since 2004 are presented in Table 2-2. Plant operations and maintenance costs are typically classified as either fixed or variable. The fixed costs remain fairly constant except for minor variations that are within typical budgeting allowances. The greatest fixed cost for a WTP is usually labor and administrative support. Variable costs are based on the annual volume of water being treated and pumped, and can also be affected by variations in raw water quality which can change chemical and solids handling requirements. The greatest variable costs for a WTP include power for pumping raw and treated water, chemicals, and solids handling and disposal. The operations and maintenance costs for the WTP have increased significantly since the 2004 WTPFP was completed for the following reasons:

- Changes in plant operating strategy including operating for longer periods each day at lower flow rates to improve plant production efficiency
- Increases in power costs
- Increases in chemical costs
- Incorporation of a solids handling program (geobag dewatering system) beginning in 2005
- Maintenance and repair related to the fish screens and screen cleaning system
- Additional plant staff and administrative support and re-structuring of the Public Works Department accounting methods

The unit production cost of treating water, shown in Table 2-2, is currently approximately $632/MG. Given the plant's current condition, this is a relatively low cost compared to similar utilities in the Pacific Northwest.

Summary

The Grants Pass WTP has supplied water to meet the City's water demands using a daily start-and-stop operating approach in the past. The recent historic peak day plant production was 14.2 mgd in July 2009 and is well below the nominal plant capacity of 20 mgd. Generally speaking, water demands have increased approximately 2 percent per year over the last decade.

The plant has performed well with regard to finished water quality and has met the regulatory requirements for filtered water turbidity. Plant production efficiencies have greatly improved since the 2004 WTPFP, averaging over 92 percent for the past five years compared to an average of about 87 percent prior. A minimum plant production efficiency of 97 percent should still be considered the long-term goal.

By switching from alum as a primary coagulant to ACH as a primary coagulant, alum usage has decreased and lime addition is no longer needed. It may be possible that coagulation chemistry between alum and ACH or PACl can be further optimized to reduce solids

production or reduce chemical addition at the plant, or both. An optimal coagulation strategy will balance plant efficiency with coagulation chemical costs, disinfection requirements, pH adjustment requirements, and residual solids production.

Table 2-2
Summary of Annual Water Treatment Plant Operations and Maintenance Costs[1]

	FY[2] 2004	FY 2005	FY 2006	FY 2007	FY 2008	FY 2009	FY 2010	FY 2011	FY 2012
Support Services									
Personnel	156,036	236,523	186,944	213,190	247,205	277,102	270,541	282,022	277,896
Operating Supplies	15,866	19,537	9,681	10,515	14,253	27,395	11,591	25,960	18,804
Contract Services	32,157	90,459	98,191	100,155	107,054	121,510	109,758	125,677	117,264
Capital Outlay	1,243	7,370	6,582	10,054	2,405	4,035	10,412	7,782	2,164
Sub-Total	*205,302*	*353,889*	*301,398*	*333,914*	*370,917*	*430,043*	*402,301*	*441,441*	*416,127*
Process									
Personnel	119,341	108,884	117,960	119,222	145,379	145,289	135,748	135,722	152,277
Operating Supplies	90,094	84,764	116,723	134,710	147,763	153,991	179,473	145,418	172,886
Contract Services	173,973	162,840	168,535	191,893	176,491	192,053	193,487	200,797	202,270
Capital Outlay	0	7,099	5,119	16,551	10,418	2,410	5,420	3,413	0
Sub-Total	*383,408*	*363,587*	*408,337*	*462,376*	*480,051*	*493,743*	*514,127*	*485,351*	*527,433*
Maintenance									
Personnel	36,613	64,256	61,188	60,758	102,155	98,076	125,817	137,101	133,999
Operating Supplies	44,421	21,152	27,197	32,744	33,402	30,492	26,727	26,877	31,815
Contract Services	26,137	24,541	22,603	40,971	43,874	53,874	70,302	41,354	19,449
Capital Outlay	10,576	0	0	5,532	1,590	4,241	0	7,735	0
Sub-Total	*117,747*	*109,948*	*110,988*	*140,004*	*181,021*	*186,683*	*222,846*	*213,067*	*185,263*
Pump Stations									
Personnel	8,485	3,967	16,686	23,989	15,787	22,630	27,795	33,598	39,667
Operating Supplies	12,159	15,159	21,259	22,000	20,651	6,855	12,341	18,660	16,150
Contract Services	78,331	81,387	77,448	98,106	88,769	99,206	107,464	129,153	115,897
Capital Outlay	5,781	292	0	1,260	0	0	0	0	0
Sub-Total	*104,756*	*100,804*	*115,393*	*145,354*	*125,207*	*128,691*	*147,599*	*181,410*	*171,714*
Solids Handling									
Personnel	143	12,416	16,915	40,095	11,831	32,734	22,652	12,838	17,908
Operating Supplies	1,364	24,111	32,895	39,229	18,617	37,382	28,761	32,187	34,245
Contract Services	2,811	11,328	30,127	15,642	18,936	17,341	29,077	38,872	30,597
Capital Outlay	0	0	801	0	0	0	0	0	0
Sub-Total	*4,319*	*47,855*	*80,738*	*94,966*	*49,384*	*87,458*	*80,490*	*83,897*	*82,749*
Total Annual Cost	**$815,532**	**$976,083**	**$1,016,855**	**$1,176,614**	**$1,206,580**	**$1,326,618**	**$1,367,363**	**$1,405,166**	**$1,432,162**
Total Annual Cost of Treatment[3]	**$698,957**	**$860,810**	**$888,959**	**$999,123**	**$1,066,960**	**$1,187,241**	**$1,203,932**	**$1,204,826**	**$1,209,408**
ADD (MGD)	**4.92**	**4.84**	**4.95**	**5.81**	**5.62**	**5.40**	**5.26**	**5.15**	**5.24**
Unit Treatment Cost ($/MG)	**$390**	**$487**	**$492**	**$471**	**$521**	**$602**	**$627**	**$641**	**$632**

Notes
1. All costs are in respective fiscal year dollars.
2. Fiscal year represented by the year at the end of the reporting period; e.g. FY 2004 represents July 2003 through June 2004.
3. Total Annual Cost of Treatment excludes Pump Station line items and all Capital Outlay costs.

Overall, the sedimentation basins provide satisfactory clarified water for filtration, as well as adequate contact time for disinfection during most of the year. All basins experience challenges with regard to short-circuiting, moderate solids loading, sub-optimal flocculation, and seasonal turbidity spikes. The basins are not equipped with any type of automated solids removal system. As solids accumulate in the basin, the effective volume of the basin is reduced, compromising flow characteristics and overall performance in the basin.

The filters have provided finished water with acceptable turbidity levels. Filtration efficiency has been improved by recent upgrades to the filters. Overall efficiency is consistently between 85 and 95 percent. Additional improvements to clarification could potentially result in increased efficiency.

As water demands continue to increase, the annual plant operating strategy may also need to be adjusted. Longer operating periods during the spring and fall months may be required. Due to occasional challenges in meeting disinfection requirements, mostly during winter cold water conditions, it may be necessary to operate the plant at lower flow rates and extend the hours of operation. Plant staffing assignments may need to be adjusted to accommodate this. These potential staffing adjustments need to be considered by the City when developing future operations budgets.

The WTP operating costs have increased by approximately 50 percent since 2004 for a variety of reasons. When considering future capacity expansions, the operating costs need to be evaluated carefully in addition to capital costs.

CHAPTER 3
REGULATORY REVIEW

Introduction

This chapter provides a general overview of current drinking water regulations under the Oregon Drinking Water Quality Act (OAR 333-061 – Rules for Public Water Systems), as well as anticipated future regulations. The discussion of each regulation is followed by an assessment of historic compliance, or in the case of future regulations, anticipated compliance. Recommended process and monitoring improvements to ensure continued compliance with all existing and anticipated regulatory requirements are discussed where appropriate. This regulatory summary is current as of April 2013. The City WTP is rated by the OHA as a conventional filtration plant. The WTP has been able to successfully produce water that has met all past and current drinking water regulations and also has met the needs of the City of Grants Pass customers.

Review of Current and Future Regulations

Currently enforced national drinking water regulations that have implications for Grants Pass are listed below:

- National Primary Drinking Water Regulations (1975)
- National Secondary Drinking Water Regulations (Secondary Standards) (1979, 1991)
- Phase I, II, and V Regulations for inorganic contaminants, synthetic organic compounds, and volatile organic compounds (1987, 1991, 1992, respectively)
- Surface Water Treatment Rule (1989)
- Interim Enhanced Surface Water Treatment Rule (1999)
- Long Term 2 Enhanced Surface Water Treatment Rule (2006)
- Total Coliform Rule (1989)
- Lead and Copper Rule (1991); being amended in 2013
- Consumer Confidence Reports Rule (1998)
- Stage 1 Disinfectants/Disinfectant By-Product Rule (1998) – supersedes Total Trihalomethane Rule (1979)
- Stage 2 Disinfectants/Disinfectant By-Product Rule (2006)
- Unregulated Contaminants Monitoring Rule 1 (1999) and 2 (2006) and 3 (2012)
- Radionuclides Rule (2000)
- Arsenic Rule (2001)
- Filter Backwash Recycle Rule (2001)

With the exception of the Unregulated Contaminants Monitoring Rule (UCMR), the water quality standards established under these national regulations have been or are planned to be adopted into the Oregon Drinking Water Quality Act (OAR 333-061) by the OHA Drinking Water Program. In addition to implementation, OHA is responsible for enforcing these

national water quality standards. If a system is found to be in violation, OHA will issue a Notice of Violation. If violations are accumulated, the system is considered a "significant non-complier." An administrative order is issued for monitoring violations or a remedial order is issued where plant improvements are required. A schedule for compliance is included in the order. If the schedule is not met, civil penalties are issued, usually in the form of fines. Enforcement of the UCMR is the responsibility of the U.S. EPA.

There are currently drinking water quality standards for 95 primary and 12 secondary contaminants in the State of Oregon (State). Under the Oregon Drinking Water Quality Act, each contaminant has either an established MCL or recommended treatment technique. These contaminants are grouped into the following general categories:

- Inorganic Contaminants,
- Organic (Synthetic and Volatile) Compounds,
- Radiologic Contaminants,
- Disinfectants and Disinfection Byproducts,
- Microbial Contaminants, and
- Secondary Contaminants.

Table 3-1 summarizes the primary and secondary drinking water contaminants regulated under the Oregon Drinking Water Quality Act found in Oregon Administrative Rule 333-061-0030. Some contaminants have a recommended treatment technique in lieu of an MCL. The following is a discussion of these state-regulated contaminants, as well as the federally monitored unregulated contaminants.

Table 3-1
Maximum Contaminant Levels and Action Levels

Contaminant	MCL[1]	Sampling Frequency
Inorganic Contaminants		
Antimony	0.006	Annually
Arsenic	0.01	Annually
Asbestos (fibers >10µm)	7 MFL	9 years
Barium	2.0	Annually
Beryllium	0.004	Annually
Cadmium	0.005	Annually
Chromium (total)	0.1	Annually
Copper	1.3[2]	See text
Cyanide	0.2	Annually
Fluoride	4.0	Annually
Lead	0.015[2]	See text
Mercury	0.002	Annually

Table 3-1 (continued)

Nickel	0.1[3]	Annually
Nitrate (as N)	10.0	Annually
Nitrate and Nitrite (as N)	10.0	Annually
Nitrite (as N)	1.0	Annually
Selenium	0.05	Annually
Thallium	0.002	Annually
Synthetic Organic Compounds		
Acrylamide	TT	Annually, if applicable
Alachlor	0.002	Twice in 3 years
Atrazine	0.003	Twice in 3 years
Benzo(a)pyrene (PAHs)	0.0002	Twice in 3 years
Carbofuran	0.04	Twice in 3 years
Chlordane	0.002	Twice in 3 years
2,4-D	0.07	Twice in 3 years
Dalapon	0.2	Twice in 3 years
Di (2-ethylhexyl) adipate	0.4	Twice in 3 years
Di (2-ethylhexyl) phthalate	0.006	Twice in 3 years
Dinoseb	0.007	Twice in 3 years
Diquat	0.02	Twice in 3 years
Endothall	0.1	Twice in 3 years
Endrin	0.002	Twice in 3 years
Epichlorohydrin	TT	Annually, if applicable
Ethylene dibromide (EDB)	0.00005	Twice in 3 years
Glyphosate	0.7	Twice in 3 years
Heptachlor	0.0004	Twice in 3 years
Heptachlor epoxide	0.0002	Twice in 3 years
Hexachlorobenzene	0.001	Twice in 3 years
Hexachlorocyclopentadiene	0.05	Twice in 3 years
Lindane	0.0002	Twice in 3 years
Methoxychlor	0.04	Twice in 3 years
Oxamyl (Vydate)	0.2	Twice in 3 years
Pentachlorophenol	0.001	Twice in 3 years
Picloram	0.5	Twice in 3 years
Polychlorinated biphenyls (PCBs)	0.0005	Twice in 3 years
Simazine	0.004	Twice in 3 years
2,3,7,8-TCDD (Dioxin)	0.00000003	Risk dependent
Toxaphene	0.003	Twice in 3 years
2,4,5-TP (Silvex)	0.05	Twice in 3 years
Volatile Organic Compounds		
Benzene	0.005	3 years
Carbon tetrachloride	0.005	3 years
1,2-Dibromo-3-chloropropane (DBCP)	0.0002	3 years

Table 3-1 (continued)

p-Dichlorobenzene	0.075	3 years
o-Dichlorobenzene	0.6	3 years
1,2-Dichloroethane	0.005	3 years
1,1-Dichloroethylene	0.007	3 years
cis-1,2-Dichloroethylene	0.07	3 years
Chlorobenzene	0.1	3 years
Dichloromethane	0.005	3 years
1,2-Dichloropropane	0.005	3 years
Ethylbenzene	0.7	3 years
Styrene	0.1	3 years
Tetrachloroethylene (PCE)	0.005	3 years
Toluene	1	3 years
1,2,4-Trichlorobenzene	0.07	3 years
1,1,1-Trichloroethane	0.2	3 years
1,1,2-Trichloroethane	0.005	3 years
Trichloroethylene	0.005	3 years
Vinyl chloride	0.002	3 years
Xylenes (total)	10	3 years
Radionuclides		
Gross alpha	15 pCi/L	4 years
Beta particle/photon activity	4 mrem/yr	4 years
Iodine - 131	3 pCi/L	4 years
Radium-226 + 228	5 pCi/L[3]	4 years
Strontium 90	8 pCi/L	4 years
Tritium	20,000 pCi/L	4 years
Disinfectant Residuals and Disinfection Byproducts		
Bromate	0.01	Quarterly
Chlorite	1.0	Quarterly
Haloacetic Acids	0.06	Quarterly
Dichloroacetic Acid	–	–
Trichloroacetic Acid	–	–
Total Trihalomethanes	0.08	Quarterly
Bromodichloromethane	–	–
Bromoform	–	–
Chloroform	–	–
Dibromochloromethane	–	–
Microbial Contaminants		
Giardia lamblia	TT	–
Cryptosporidium	TT	–
Legionella	TT	–
Heterotrophic plate count	TT	–
Turbidity	TT	See text
Viruses	TT	–

Table 3-1 (continued)

Total Coliform (TC)	< 5% positive	40/month
Fecal Coliform	Confirmed Presence	–
E. Coli	Confirmed Presence	If Total Coliform Test Positive
Secondary Standards		
Color (Color Units)	15	–
Corrosiveness	Noncorrosive	–
Foaming Agents	0.5	–
pH	6.5 to 8.5	–
Hardness (as $CaCO_3$)	250	–
Odor	3 TON[4]	–
Total Dissolved Solids	500	–
Aluminum	0.05 to 0.2	–
Chloride	250	–
Copper	1	–
Fluoride	2.0	–
Iron	0.3	–
Manganese	0.05	–
Silver	0.1	–
Sulfate	250	–
Zinc	5.0	–

Notes
1. Values reported in mg/L unless otherwise specified.
2. Action Level
3. MCL currently being re-evaluated by the EPA
4. Threshold odor number

Surface Water Treatment

All public water systems using surface water sources are required to comply with the Oregon Drinking Water Quality Act's treatment performance and disinfection requirements. Three specific areas are addressed within the Act, including:

- Overall filtration performance,
- Individual filtration performance, and
- Disinfection performance.

These are discussed below in detail.

Overall Filtration Performance Requirements

Current overall filtration performance standards require that the turbidity measurements from the combined filter effluent must be measured in 4-hour intervals by grab sampling or continuous monitoring. Ninety-five percent of these turbidity readings must be less than or equal to 0.3 NTU, and may never exceed 1.0 NTU. In addition, treatment strategies, in

combination with disinfection, must consistently remove or inactivate 99.9 percent (3-log) of *Giardia*, 99.99 percent (4-log) of viruses, and 99 percent (2-log) removal (i.e., no inactivation) of *Cryptosporidium*. Each utility is required to submit a report to the State on a monthly basis and identify any exceptions.

Individual Filter Performance Requirements

Oregon law requires continuous, on-line measurement of turbidity for each individual filter. This data must be recorded every 15 minutes. If there is a failure in the turbidity monitoring equipment, the system may conduct grab sampling every 4 hours in lieu, but for not more than 5 working days following the failure. Each utility is required to submit a report to the State on a monthly basis and identify any exceptions. Exceptions under Oregon law occur when:

1. Individual filter effluent turbidity exceeds 1.0 NTU in two consecutive measurements, 15 minutes apart at any time during the filter operation.
2. Individual filter effluent turbidity exceeds 0.5 NTU in two consecutive measurements, 15 minutes apart, after 4 hours of operation following backwash.
3. Individual filter effluent turbidity exceeds 1.0 NTU in two consecutive measurements, 15 minutes apart, at any time during the filter operation in three consecutive months or for three months in a row.
4. Individual filter effluent turbidity exceeds 2.0 NTU in two consecutive measurements, 15 minutes apart, at any time during the filter operation in two consecutive months or for two months in a row.

Disinfection Performance Requirements

The Oregon Drinking Water Quality Act requires all utilities served by a surface water supply to achieve a minimum of 99.9 percent (3-log) reduction in *Giardia lamblia* cysts, 99.99 percent (4-log) reduction in viruses, and 99 percent (2-log) removal of *Cryptosporidium* cysts during drinking water treatment. Removal credit is awarded to WTPs based on the types of processes provided by the plants. For a conventional filtration plant with filter-to-waste capabilities, such as the Grants Pass WTP, a 2.5-log, 2.0-log, and 2.0-log removal credit is usually granted for *Giardia lamblia*, viruses, and *Cryptosporidium*, respectively. The remaining reduction in pathogenic organisms must come in the form of disinfection or inactivation, or both. For the Grants Pass WTP, a minimum of 0.5-log inactivation of *Giardia* and 2.0-log inactivation of viruses is required prior to the first customer. Due to its longer time requirement for inactivation, *Giardia* inactivation typically governs disinfection through the WTP compared to viruses.

To determine the level of inactivation achieved during chemical disinfection, the EPA developed the "CT" concept. "CT" is the product of disinfectant residual measured at the outlet of a disinfection section and the time in which 10 percent (by volume) of an added tracer passes through the section, known as the T_{10}. To remain in compliance with disinfection performance standards, the following criteria must be met:

1. Disinfection residual must be continuously recorded at the entry point to the distribution system and must never fall below 0.2 mg/L.
2. CT must be calculated every day. To ensure that the values are conservative, the highest flow rate and minimum clearwell volume recorded for the day must be used in the calculation; tracer studies should be used to verify hydraulic efficiencies through the various treatment trains.
3. The CT calculated must be sufficient to meet the needed removal or inactivation levels.
4. The residual disinfectant concentration in the distribution system cannot be undetectable in more than 5 percent of the samples. For simplicity, samples should be collected at coliform bacteria monitoring points.

In Oregon, the OHA also enacted a requirement in the mid 1990s that a minimum of 0.5-log inactivation of *Giardia* and 1.0-log inactivation of viruses must be achieved following filtration and prior to the first customer. The OHA has grandfathered the Grants Pass WTP and allowed a disinfection credit for pre-chlorination through the plant upstream of the clearwell, including basins before filtration and the filters themselves. The City has been proactive in communicating the disinfection profile at the plant to the OHA and has worked with the State to ensure that the evaluation of CT at the plant is accurate. The rating and status of the WTP should remain the same as long as the WTP continues to meets water quality requirements and there are no major projects completed that would alter plant performance. In addition, the plant will be limited to a maximum capacity of 20 mgd. If flow exceeds this limit on a filter-by-filter basis, the WTP status will be reviewed and the ability to count pre-filtration CT could be revoked. In most cases, the OHA offers no disinfection credit for conventional plants prior to filtration even if a chlorine residual is carried through the unit operations preceding filtration.

Historical Compliance

The Grants Pass WTP complies with the Oregon Drinking Water Quality Act. Performance is discussed in the sections that follow.

Overall Filter Performance

Filtered water turbidity is measured at the combined filter effluent before entering the clearwell in the filter gallery. During the period from January 2004 to December 2011, filter effluent turbidity averaged 0.03 NTU. No filter effluent samples during this period exceeded the regulatory maximum of 1.0 NTU. The WTP has been in compliance with this regulation for the past 7 years.

Individual Filter Performance

On-line turbidimeters necessary for monitoring the individual filtered water turbidity have been used at the WTP for many years. Plant staff indicated that none of the individual filter

effluent turbidity thresholds have ever been exceeded since their installation in the early 1990s.

Disinfection Performance

CT achieved through the WTP and through the clearwell is calculated daily according to OHA's guidelines which were originally established in the mid 1990s. Calculations include the daily finished water temperature, chlorine residual of the basin effluent and clearwell effluent measured every hour, pH, and the maximum daily treated water flow. Once calculated, this value is compared to the CT required; if CT achieved is greater than the CT required, then compliance is achieved.

The actual CT value is currently being calculated from a tracer study that was completed in 2003 for the clearwell. A new tracer study was recently completed to verify the T_{10}/T assumed for the WTP upstream of the clearwell. Appendix A includes a detailed summary of the new tracer study. To date, Grants Pass has consistently met CT requirements at the WTP using the calculation methodology approved by OHA. From 2009 to 2011, there was only one instance where the total calculated *Giardia* inactivation through the plant was less than 0.5-log. This occured on October 29, 2009 when a value of 0.49 log was recorded. The WTP has had no violations with regard to disinfection residual monitoring or residual concentrations in the distribution system. Calculated CT values through the plant from year 2009 to 2011 are shown in Figure 3-1. Figure 3-1 also shows the plant's internal benchmark of 0.75-log *Giardia* inactivation. This benchmark is normally achieved except during the spring and fall seasons and during periods with very low raw water temperature.

If OHA decides to change the calculation methodology used by the plant to only allow credit for CT achieved through the clearwell, the plant may be significantly challenged to meet the CT required throughout the year. According to Figure 3-2, from 2009 to 2011, the CT achieved in the clearwell did not result in 0.5-log *Giardia* inactivation for almost 10 percent of the time. This often occurred during the winter months when the water temperature was very cold or during the spring and fall seasons when water demand started to increase, resulting in a higher plant operating flow rate, while the water temperature was still fairly cold. It is likely that the plant could modify its operational procedures during challenging water quality periods to be able to achieve 0.5-log *Giardia* inactivation in the clearwell under all conditions, mostly by operating the plant at a lower flow rate for longer periods.

If the WTP were ever to be rated by OHA as a direct filtration plant instead of a conventional filtration plant, then it would have to achieve a minimum of 1.0-log *Giardia* inactivation through the plant and at least 0.5-log *Giardia* inactivation would have to be achieved post-filtration. As seen from Figure 3-1, achieving 1.0-log *Giardia* inactivation throughout the year would be extremely challenging and may not be possible without significant capital improvements.

Figure 3-1
Overall *Giardia* Inactivation Achieved

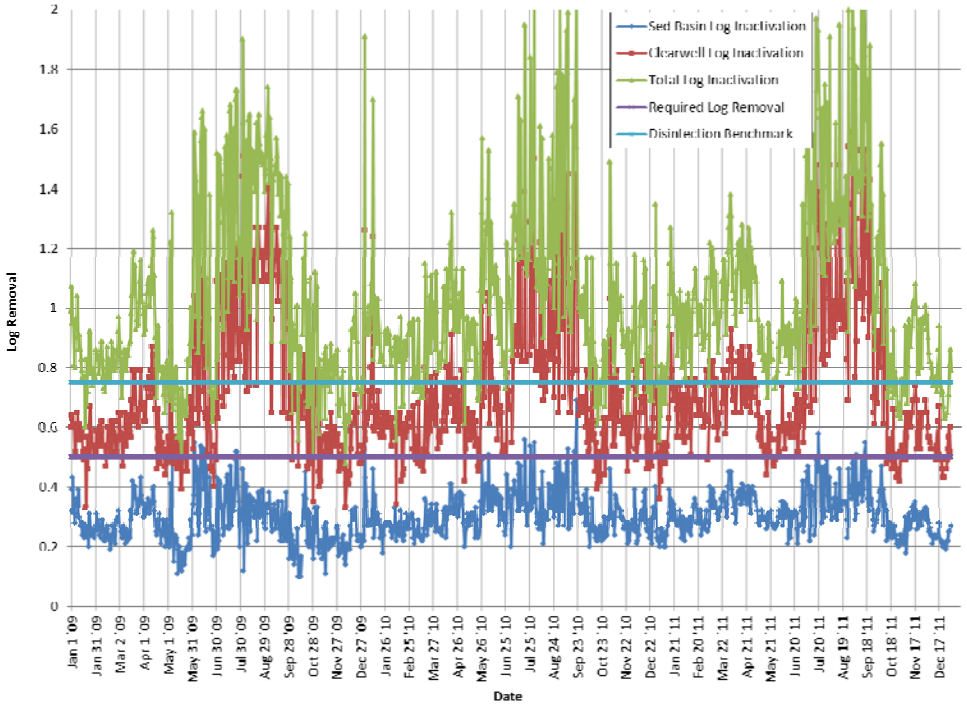
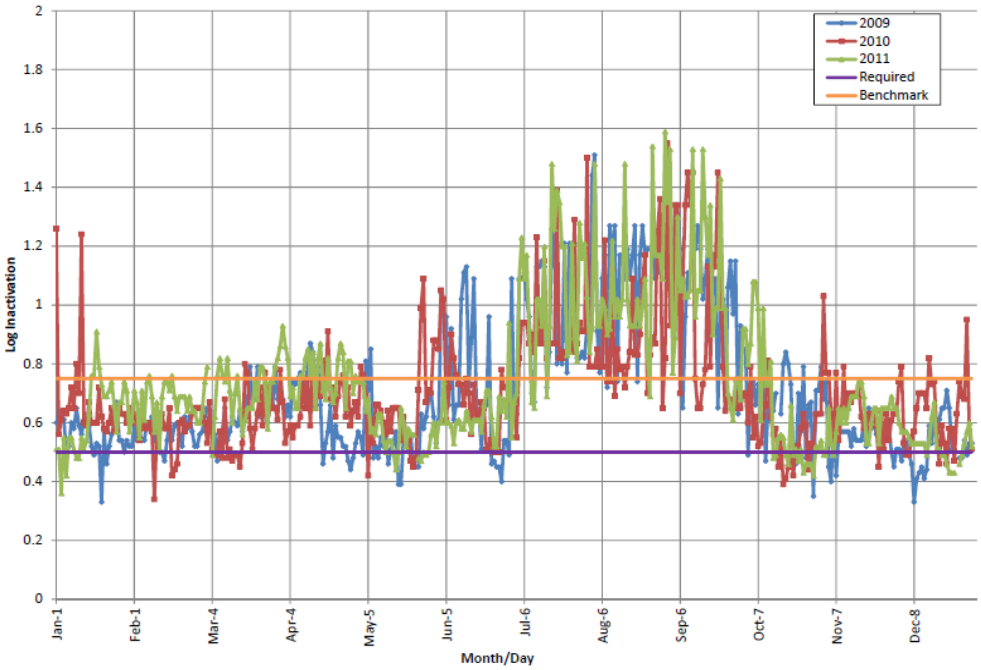


Figure 3-2
***Giardia* Inactivation Through the Clearwell**



Total Coliform Rule

Based on the City's population, the Oregon Drinking Water Quality Act requires the City of Grants Pass to collect a minimum of 40 water samples per month from representative sites throughout the water distribution system. If a routine sample is positive for total coliform, the City must collect a set of three repeat samples: one from the original site, one from a location within five service connections upstream of the original site, and one from a location within five service connections downstream of the original site.

The repeat samples must be collected within 24 hours of notification of the positive result. Further, any routine or repeat coliform positive samples must be analyzed for the presence of fecal coliform or *E. coli* as an indicator organism. When a system learns of the presence of fecal coliform or *E. coli*, the system must notify the State by the end of the same day. In Oregon, the total coliform MCL is violated in any of the following situations:

1. More than one sample collected within a single month is coliform positive, referred to as a non-acute violation.
2. A repeat sample following a total coliform positive contains fecal coliform or *E. coli*, referred to as an acute violation.
3. A repeat sample following a fecal coliform positive or *E. coli* positive contains total coliform, also an acute violation.

The City of Grants Pass monitors all of the water system microbial data, since the City owns and operates its distribution system that receives water produced by the WTP. To date, no information has been identified that indicates the City has violated the Total Coliform Rule. The finished water produced by the WTP has always met the requirements related to maintaining the minimum chlorine residual and booster chlorination is practiced in the distribution system at key locations to ensure that a minimum residual is maintained.

Long-term 2 Enhanced Surface Water Treatment Rule

The purpose of the Long-term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) is to further improve the control of *Cryptosporidium* in drinking water. The LT2ESWTR was published in the *Federal Register* on January 5, 2006. It applies to public water systems serving 10,000 or more people. Compliance with the LT2ESWTR was required in 2008 for the Grants Pass WTP. The LT2ESWTR requirements that potentially will impact the Grants Pass WTP include:

1. Source water sampling to establish concentrations of *Cryptosporidium*, which in turn defines additional treatment requirements for *Cryptosporidium*.
2. Potential additional *Cryptosporidium* inactivation and removal requirements.
3. Incorporation of a multi-barrier disinfection strategy.

To quantify system vulnerability, a 24-month monitoring program for *Cryptosporidium* is required to classify plants into treatment bins associated with source water concentration.

The rule includes a "toolbox" of control measures for meeting treatment requirements including watershed control options, treatment options, filter performance, and challenge tests. Table 3-2 presents the proposed treatment requirements for conventional plants and direct filtration plants based on results from the monitoring program.

Table 3-2
LT2ESWTR *Cryptosporidium* Monitoring Bin Classifications

Bin Number	Sample Results (Crypto oocyst per liter raw water)	Additional Treatment Requirements
1	< 0.075	No additional treatment required
2	0.075 to 1.0	1-log reduction
3	1.0 to 3.0	2-log reduction (1-log from disinfection)
4	> 3.0	2.5-log reduction (1-log from disinfection)

Non-disinfection-related reduction can be achieved through one or more alternatives presented in the LT2ESWTR "toolbox", below.

- Watershed control – 0.5 log.
- Alternative source or intake management – can get lower bin assignment.
- Off-stream storage – 0.5 log, 1.0 log based on hydraulic residence time.
- Pre-sedimentation basin (with coagulation) – 0.5 log
- Lime softening – 0.5 log
- Lower finished water turbidity – 0.5 log for CFE of 0.15 NTU (95 percent of the time), or 1.0 log for individual filter effluent less than or equal to 0.15 NTU (95 percent of the time). Cannot get credit for both.
- Membranes – Demonstrated with integrity testing for membranes that have been challenge-tested by the manufacturer.

In addition to raw water monitoring requirements, the LT2ESWTR requires all systems to perform disinfection profiling. If any modifications are made to the WTP, the WTP will need to work with OHA to establish expectations for the disinfection profile for the plant improvements.

The Rogue River is classified as a Bin #1 supply by OHA and therefore does not require any additional treatment processes for *Cryptosporidium* inactivation or removal. Extensive testing has been done on the Rogue River to validate this classification.

Disinfectants and Disinfection Byproducts

The Federal Total Trihalomethane Rule (TTHM Rule) was published in the *Federal Register* in November 1979; Oregon adopted the MCLs established in this law in September 1982.

The TTHM Rule set an MCL for TTHM of 0.10 mg/L based on a running annual average of quarterly sampling in the distribution system. However, these MCLs were superseded when the State of Oregon adopted the Stage 1 Disinfectants/Disinfection Byproducts Rule (D/DBPR) on July 15, 2000. The Stage 1 D/DBPR added an MCL of 0.060 mg/L for five haloacetic acids (HAA5), and reduced the MCL for TTHMs to 0.080 mg/L. The Stage 2 D/DBPR was promulgated by the EPA on January 4, 2006 and built on the Stage 1 rule by requiring that compliance be based on locational running annual averages (LRAAs) rather than a system-wide average of all sample locations. In addition, the Stage 2 D/DBPR required systems to revisit sample locations and perform more DBP sampling to determine sample locations that are most representative of worst-case DBP water quality. According to the OHA guidelines, the City's schedule for meeting the Stage 2 D/DBP Rule is as follows:

- 10/1/2007: Submit IDSE standard monitoring plan
- 9/30/2009: Complete an initial distribution system evaluation
- 1/1/2010: Submit IDSE report
- 10/1/2013: Begin Stage 2 compliance monitoring

To date, the City has completed the first three tasks and has now begun preliminary sampling of its stage 2 sites in preparation for Stage 2 compliance monitoring.

Monitoring Requirements

The Oregon Drinking Water Quality Act requires monitoring of disinfection byproducts. Compliance is currently based on a system-wide running annual average of quarterly samples, but in 2013 will move to a locational running annual average at each of the four sampling locations. To remain in compliance, the locational running annual average for TTHMs and HAA5s must not exceed 0.08 mg/L and 0.060 mg/L, respectively, at any location. Table 3-3 shows the DBPs and corresponding MCLs.

Table 3-3
Maximum Contaminant Levels for Disinfection Byproducts

Contaminant	Maximum Contaminant Level (mg/L)
Total Trihalomethanes[1] (TTHMs)	0.080
Haloacetic Acids[2] (HAA5)	0.060

Notes
1. "Total Trihalomethanes" includes the sum of concentrations of chloroform, bromodichloromethane, dibromochloromethane, and bromoform.
2. "Haloacetic acids" includes the sum of concentrations of monochloroacetic, dichloroacetic, trichloroacetic, monobromoacetic, and dibromoacetic acids.

Maximum residual disinfectant levels (MRDLs) present in the distribution system are also regulated. These MRDLs are summarized in Table 3-4. Monitoring and compliance for the MRDL of chloramines is similar to that required under the Total Coliform Rule (TCR). Utilities are required to collect these disinfection residual samples at the same locations and frequency as coliform samples.

Table 3-4
Maximum Residual Disinfectant Levels

Disinfectant	Maximum Residual Disinfectant Level (mg/L)
Chlorine	4.0 mg/L as Cl_2
Chloramines	4.0 mg/L as Cl_2
Chlorine Dioxide	0.8 mg/L as ClO_2

In addition to DBP MCLs and disinfectant MRDLs, conventional WTPs that have surface water as a supply are required to remove specific amounts of organic material through their treatment process. The percent of removal required depends on source water TOC and alkalinity. Table 3-5 provides a summary of the removal requirements.

Table 3-5
Percent Required Removal of Total Organic Carbon by Enhanced Coagulation for Plants Using Conventional Treatment

Total Organic Carbon in Raw Water (mg/L)	Source Water Alkalinity (mg/L as $CaCO_3$)		
	0 – 60	60 – 120	> 120
2.0 – 4.0	35	25	15
4.0 – 8.0	45	35	25
> 8.0	50	40	30

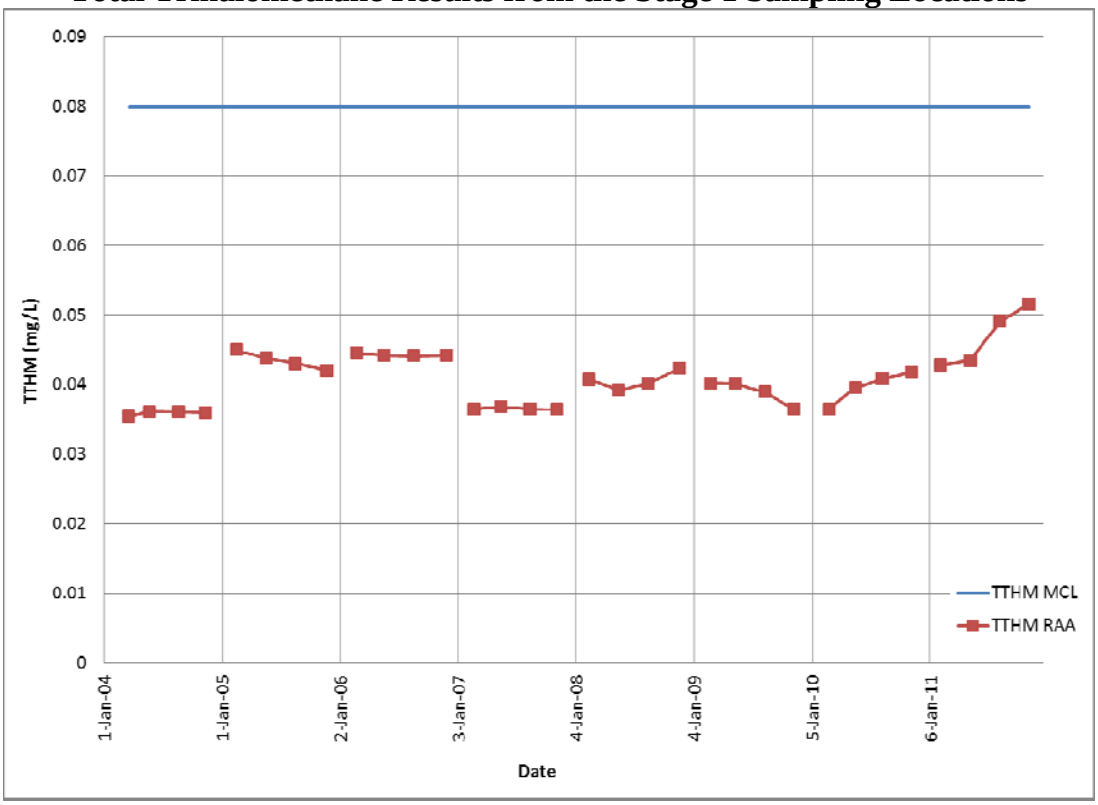
Compliance with this treatment requirement must be calculated as a running annual average on a quarterly basis after 12 months of data are available. Systems having raw water TOC concentrations under 2.0 mg/L are exempt from any TOC removal requirements.

Historical Compliance and Implications for Future Operation

The City of Grants Pass samples for the regulated DBPs at various locations throughout the distribution system. The current sampling protocol for DBPs includes four sites, with one sample representative of the maximum residence time in the distribution system at the Merlin Landfill and the remaining sample locations at the New Hope Pump Station, the Water Restoration Plant, and the Hillcrest Fire Station. The latter three sites are representative of the average residence time through the distribution system. Stage 2 protocol will add three additional sampling sites.

Prior to 2010, the City was only required to take four samples per quarter and that data was used to calculate a RAA for the average of the four samples. Figures 3-3 and 3-4 present DBP monitoring data for TTHMs and HAA5s prior to 2010 which was used to determine compliance with the Stage 1 D/DBP rule.

Figure 3-3
Total Trihalomethane Results from the Stage 1 Sampling Locations



Compliance with Stage 2 D/DBP will require a locational running annual average approach at each of the seven sampling locations and will be determined based on the worst-case location in the distribution system. It is highly likely that the worst-case location for TTHMs and HAA5s will be at the Merlin Landfill. However, due to HAA5s being mainly formed immediately downstream of the clearwell, the LRAA for HAA5s could be in a different location. Figures 3-5 and 3-6 present the LRAA TTHMs and HAA5s monitoring data from 2004 to 2011 at the Merlin Landfill sampling location.

Based on the historical DBP monitoring data, there have been periods when both TTHMs and HAA5s have been elevated above the regulatory limits, but no violations of the Stage 1 D/DBP Rule have occurred. There is no consistent annual pattern of elevated DBPs that would suggest that dramatic changes would have to be made to the treatment process. It is also not clear what influence raw water TOC and TOC removal through the plant has on DBP formation. It is possible that additional plant operating improvements or optimized distribution system operations may be able to ensure compliance with the future Stage 2

Figure 3-4
Haloacetic Acids Results from the Stage 1 Sampling Locations

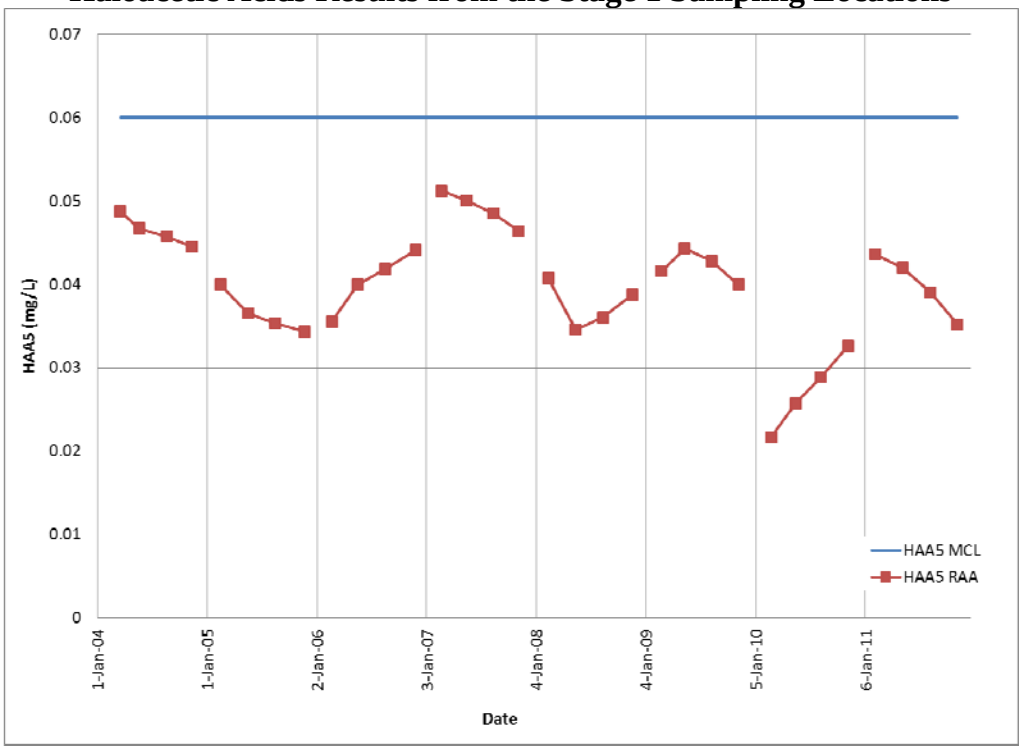


Figure 3-5
Total Trihalomethane Results at the Merlin Landfill

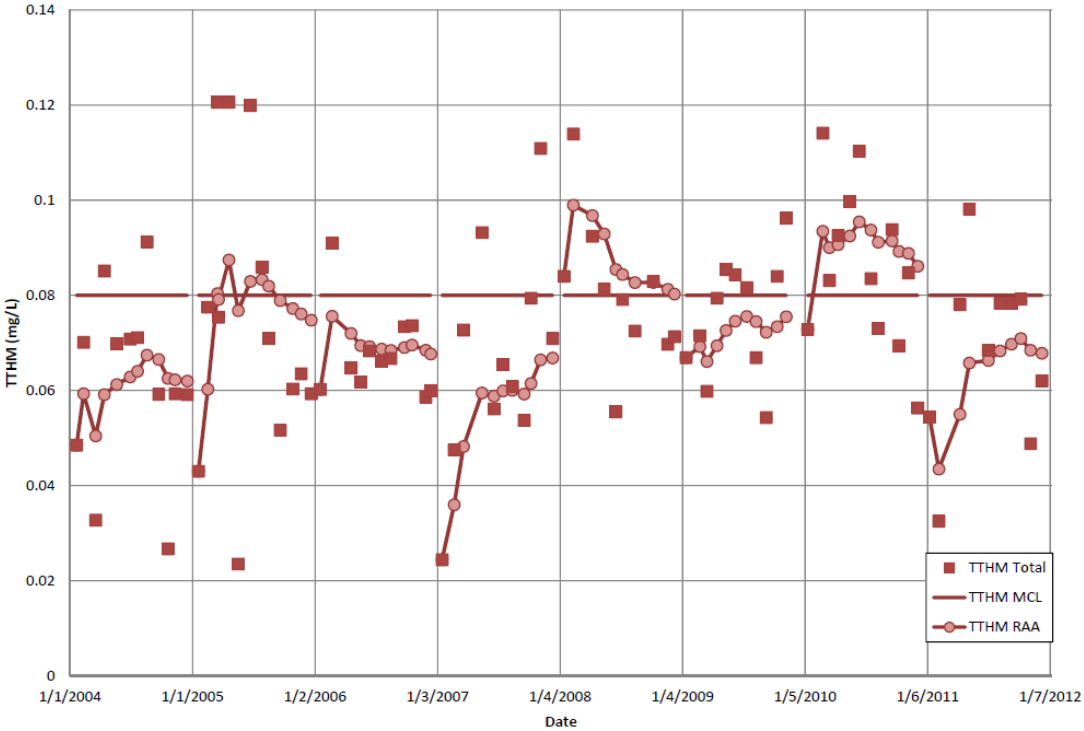
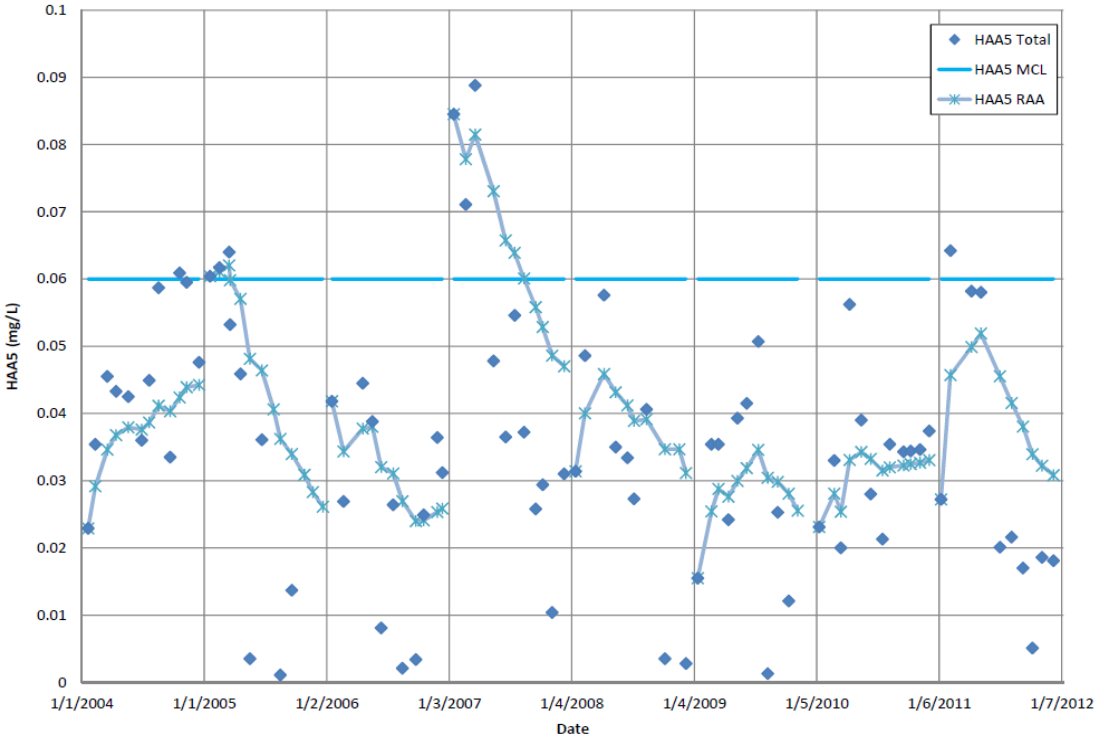


Figure 3-6
Haloacetic Acids Results at the Merlin Landfill



D/DBP Rule. These improvements could be lowering the chlorine residual in the plant and decreasing residence time in the distribution system.

Total Organic Carbon

The Grants Pass WTP monitored raw and finished water TOC monthly from 2004 to 2011 and this data is presented in Figure 3-7. Since the RAA of the raw water TOC was less than 2.0 mg/L, the City is not required to achieve a regulated amount of TOC removal through the plant. Also, the plant has recently had its TOC sampling frequency reduced from monthly to quarterly. The average raw TOC concentration in the Rogue River source from 2004 to 2011 was 1.6 mg/L and historical TOC removal through the plant has averaged 35 percent on an annualized basis. Unless the quality of the source water drastically changes, it is unlikely that TOC removal will be a problem for the Grants Pass WTP.

Lead and Copper and Corrosion Control

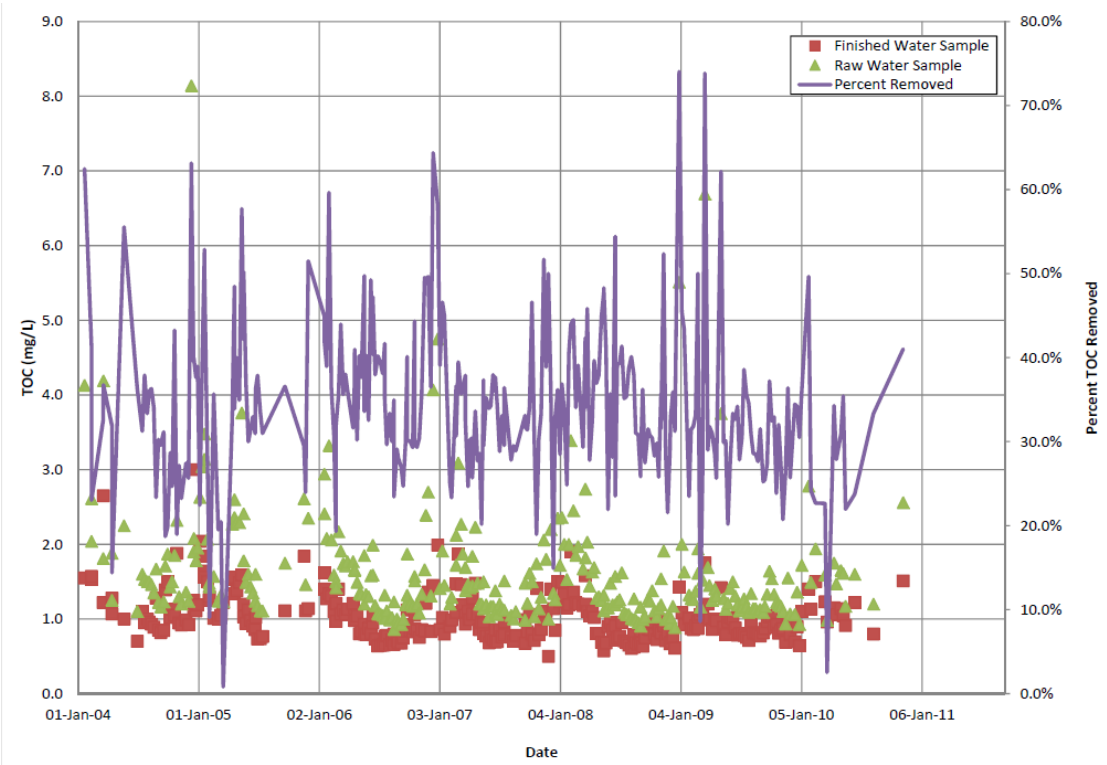
In 1991, LCR was promulgated by the EPA to reduce lead and copper concentrations in drinking water. Oregon adopted the LCR on December 7, 1992, without exception. The Lead and Copper rule established action levels for lead and copper set at 0.015 mg/L and 1.3 mg/L, respectively. Lead and copper regulations, under the Oregon Drinking Water Quality Act, require utilities to implement optimal corrosion control treatment that minimizes the lead and copper concentrations at users' taps, while ensuring that the treatment efforts do not

cause the water system to violate other existing water regulations. It should be noted that an update to the LCR is expected to be promulgated in 2013, though implications to the City's plant are anticipated to be minimal.

Monitoring Requirements

Utilities are required to conduct monitoring for lead and copper from taps in "high risk" homes. Two rounds of initial sampling were required from 1992 to 1994, collected at 6-month intervals. Annual sampling was required after these initial efforts. Following this initial three-year period of sampling, samples are to be taken every three years. The action level for either compound is exceeded when, in a given monitoring period, more than 10 percent of the samples are greater than the action level.

Figure 3-7
Raw and Finished Water Total Organic Carbon Concentrations and Percent Removal



Sampling requirements of the LCR are based on the population served by the utility. For the service area of the Grants Pass WTP, which has a combined population of between 10,001 and 100,000, Oregon law requires 60 initial sampling sites; subsequent monitoring could be reduced to 30 sites provided initial sampling efforts demonstrate that lead and copper action levels are not exceeded. Water systems unable to meet action levels must either integrate corrosion control strategies into their treatment process train or develop an alternate source of water.

Historical Compliance

The Grants Pass WTP has historically produced non-corrosive water, keeping it in compliance with the Lead and Copper Rule since it was enacted in the early 1990s. Due to the WTPs ability to consistently produce water with low corrosiveness as evidenced by low 'at-the-tap' concentrations of lead and copper, OHA has reduced the sampling frequency to once every three years. There appears to be no concerns with future compliance with the Lead and Copper Rule.

Inorganic Contaminants

The goal of the Primary Drinking Water Regulations, with regard to inorganic contaminants, is to control the levels of minerals and metals in drinking water that create health concerns. For most inorganic contaminants, these health concerns result after long-term exposure to the compounds. However, the risks associated with nitrates and nitrites are acute; thus, additional monitoring requirements for nitrates and nitrites are included in Oregon law.

Monitoring Requirements

Monitoring requirements and MCLs for regulated inorganic contaminants are included in Table 3-1. Initial monitoring for nitrite and nitrate was quarterly for a minimum of one year. If all collected samples were below 50 percent of the MCLs for nitrite and nitrate, sampling was reduced to yearly. For water systems that contain asbestos-cement water pipes, samples testing for asbestos fibers must be taken every nine years. Monitoring for and compliance with the new arsenic MCL of 0.010 mg/L was required by January 2006. Concentrations of all other inorganic contaminants must be measured annually. Quarterly follow-up testing is required for any contaminants that are detected above the MCL.

Historical Compliance

The Grants Pass WTP has remained in compliance with regard to all inorganic contaminant MCLs during the period evaluated. Due to the high quality of the source water, the WTP is only required to sample for inorganic contaminants every nine years.

Organic Contaminants

Maximum contaminant levels for 53 different organic contaminants under the Oregon Drinking Water Quality Act were adopted from the Safe Drinking Water Act (SDWA) and are listed in Table 3-1. Monitoring requirements and MCLs for synthetic organic compounds (SOCs) and volatile organic compounds (VOCs) are listed in Table 3-1. The WTP monitors VOCs yearly and SOCs two consecutive quarters every three years per the state requirements. No concentration of regulated VOCs or SOCs above the detection limit is on record in the past five years.

Radiological Contaminants

The original MCLs adopted from the NPDWR by Oregon on September 24, 1982 are still in effect in the Oregon Drinking Water Quality Act today. These rules were revised in October 2002 to include a new MCL for Uranium and to clarify and modify monitoring requirements. Together, these established MCLs seek to minimize the cancer risk associated with long-term exposure to six natural and manmade radiological contaminants.

Monitoring Requirements

Monitoring requirements and MCLs for radiological contaminants are listed in Table 3-1. Monitoring for radionuclides is required once every four years from surface water sources. If gross alpha is measured below 5 picocuries per liter (pCi/L), no radium analyses are required. Only systems with elevated risks, such as impacts by manmade radiation sources, must sample for beta and photon radiation.

Historical Compliance

The City WTP staff analyzes radiological samples every nine years, a reduction in monitoring frequency granted by OHA based on no detection of radiological contaminants. The WTP has fully complied with all OHA radiological standards for the period evaluated, and no elevated gross alpha measurements have ever been observed.

Federally Monitored Unregulated Contaminants

The final UCMR was published by the EPA in the March 12, 2002 *Federal Register*. Under this rule, EPA develops a list of unregulated contaminants every five years. Contaminants on the list are under consideration for eventual regulation but the EPA has insufficient occurrence information for each of them. This rule is administered and enforced by the EPA rather than the State primacy agencies.

Monitoring Requirements

UCMR 1, published in 1999, established a new list of contaminants to be monitored, procedures for selecting a national representative sample of public water systems, and procedures for incorporating the monitoring results into the National Contaminant Occurrence Database. UCMR 1 re-designed the UCM program to incorporate a tiered monitoring approach that divided monitoring of contaminants into three lists:

- List 1 contaminants are monitored by all public water systems serving over 10,000 people and a smaller group of public water systems serving less than 10,000 people;
- List 2 contaminants are monitored by a representative group of 300 randomly chosen public water systems;

- List 3 contaminants are monitored by 200 “vulnerable” systems across the country.

For chemical contaminants, surface water systems monitor quarterly for one year and ground water systems monitor two times six months apart. For microbiological contaminants, systems monitor twice, six months apart. For all chemical constituents in Lists 1 and 2, monitoring must be conducted at the entry point to the distribution system. For microbiological contaminants in List 1, monitoring is conducted near the end of the distribution system and at a representative site within the distribution system. Nationwide sampling for UCMR 1 took place from 2001 to 2003. The list of UCMR 1 contaminants is provided in Table 3-6.

The second monitoring cycle established a new list of contaminants in UCMR 2, promulgated in 2007. The WTP completed its UCMR 2 monitoring, which nation-wide extended from 2008 through 2010. Twenty-five contaminants were listed by the EPA for monitoring under UCMR 2: 10 List 1 contaminants and 15 List 2 contaminants, which are shown in Table 3-6.

UCMR 3 was finalized in May 2012. The City will begin monitoring and reporting the 30 identified contaminants (28 chemical, 2 viruses) in 2013. The program will be running from 2013 to 2015 and have similar sampling and reporting requirements as UCMR 2.

Historical Compliance

The WTP has historically complied with unregulated contaminant monitoring required by the EPA. No contaminants of concern have been detected in the Rogue River supply.

Secondary Standards

The secondary standards for drinking water, listed in Table 3-1, are intended as guidelines that address water quality issues which are related to the taste, odor, aesthetics, and corrosiveness of drinking water. These standards are non-enforceable guidelines for water quality parameters not known to adversely affect human health.

The WTP monitors finished water alkalinity, pH, chlorine, and turbidity on a daily basis as presented in Chapter 2. The WTP has occasionally received customer complaints related to drinking water tastes and odors on an infrequent and seasonal basis. The Grants Pass WTP has historically complied with water regulations addressed by the Secondary Standards.

Filter Backwash Recycling Rule

The final Filter Backwash Recycling Rule (FBRR), promulgated in 2001, applies to all public water systems that use surface water and employ conventional or direct filtration and also recycle water within the plant.

Table 3-6
Unregulated Contaminant Monitoring Program Summary

Unregulated Contaminant Monitoring Rule 1		
List 1	**List 2**	**List 3**
Assessment Monitoring of Contaminants with Available Methods	**Screening Surveys of Contaminants with Methods Just Developed**	**Prescreen Testing of Contaminants Needing Research on Methods**
2,4-dinitrotoluene	1,2-diphenylhydrazine	Lead-210
2,6-dinitrotoluene	2-methyl-phenol	Polonium-210
Acetochlor	2,4-dichlorophenol	Cyanobacteria
DCPA mono-acid degradate	2,4-dinitrophenol	Echoviruses
DCPA di-acid degradate	2,4,6-trichlorophenol	Coxsackieviruses
4,4'-DDE	Diazinon	Helicobacter pylori
EPTC	Disulfoton	Microsporidia
Molinate	Diuron	Caliciviruses
MTBE	Fonofos	Adenoviruses
Nitrobenzene	Linuron	
Perchlorate	Nitrobenzene	
Terbacil	Prometon	
	Terbufos	
	Aeromonas	
	Alachlor ESA	
	RDX	

Unregulated Contaminant Monitoring Rule 2	
List 1	**List 2**
Dimethoate	**Three Parent Acetanilides**
Terbufos sulfone	Acetochlor
Five Flame Retardants	Alachlor
2,2',4,4'-tetrabromodiphenyl ether (BDE-47)	Metolachlor
2,2',4,4',5-pentabromodiphenyl ether (BDE-99)	**Six Acetanilide Degradates**
2,2',4,4',5,5'-hexabromobiphenyl (HBB)	Acetochlor ethane sulfonic acid (ESA)
2,2',4,4',5,5'-hexabromodiphenyl ether (BDE-153)	Acetochlor oxanilic acid (OA)
2,2',4,4',6-pentabromodiphenyl ether (BDE-100)	Alachlor ethane sulfonic acid(ESA)
Three Explosives	Alachlor oxanilic acid (OA)
1,3-dinitrobenzene	Metolachlor ethane sulfonic acid(ESA)
2,4,6-trinitrotoluene (TNT)	Metolachlor oxanilic acid (OA)
Hexahydro-1,3,5-trinitro-1,3,5-triazine (RDX)	**Six Nitrosamines**
	N-nitroso-diethylamine (NDEA)
	N-nitroso-dimethylamine (NDMA)
	N-nitroso-di-n-butylamine (NDBA)
	N-nitroso-di-n-propylamine (NDPA)
	N-nitroso-methylethylamine (NMEA)
	N-nitroso-pyrrolidine (NPYR)

Monitoring and Compliance Requirements

This rule requires the three major recycle streams, spent filter backwash water, solids thickener supernatant, and liquids from dewatering processes, to pass through all treatment processes. Therefore, these recycle streams must be returned prior to chemical addition and coagulation. The rule is unclear as to whether FTW water is considered a recycle stream and whether such water can be returned downstream of chemical addition and coagulation. This decision is made between the utility and OHA on a case-by-case basis.

Each utility was required to notify OHA in writing by December 8, 2003, that they practice recycling. This notification included a plant schematic that shows the type and location of recycle streams, typical recycle flow data, highest plant flow in the previous year, design flow of the plant, and OHA-approved operating capacity. Each system must collect and maintain the following information for compliance with this rule:

- Copy of recycle notice to OHA.
- List of all recycle flows and frequency.
- Average and maximum backwash flow and duration.
- Typical filter run duration and how it was determined.
- Type of recycle treatment (if any) and data on recycle stream facilities.

This rule may affect decisions regarding how recycle streams are handled for a new or upgraded WTP.

Historical Compliance

Since the WTP does not recycle any of its residual streams, the FBRR does not apply, but is mentioned for consideration if WTP operational issues drive the plant to recycle some or all of its waste streams in the future. The plant sends its filter backwash water and filter-to-waste to the old mill pond located across the street from the WTP. The old mill pond releases decant or overflow water to Skunk Creek. A NPDES permit has been issued by Oregon DEQ for this discharge stream. Solids from the sedimentation basin are dewatered on-site using geobags and the “pressate” is not recycled within the WTP.

Tastes and Odors

Taste and odor events from the City’s water supply are very rare in Grants Pass. Other upstream users of the Rogue River, such as the Medford Water Commission, experience taste and odor episodes on a frequent basis. The common taste and odor reported in Medford is earthy or musty and is commonly caused by excessive algal activity. The conditions in the lower Rogue River in and around Grants Pass are apparently not as conducive to excessive algal activity during the summer and fall as in the upper parts of the watershed. Algae can produce excessive concentrations of MIB and geosmin which are organic compounds that

impart earthy or musty tastes and odors to the water. These compounds do not present a health hazard, but create an aesthetic and public perception problem.

Because of the low historical occurrence of taste and odor in its water supply, the Grants Pass WTP is not equipped with processes capable of removing earthy or musty tastes and odors. The only treatment alternatives for this particular water quality issue include the following:

- Oxidation with ozone
- Adsorption with high doses of powdered activated carbon (PAC)
- Adsorption with granular activated carbon (GAC), either as a filter adsorber or in a separate contactor
- Oxidation using ultraviolet (UV) light combined with addition of hydrogen peroxide

The Medford Water Commission's Duff WTP uses pre-ozonation to combat earthy or musty tastes and odors. Before ozonation was installed, there was a high frequency of taste and odor events and customer complaints received when the City started up the Duff WTP in the summer to handle their peaks in demand. The rest of the year, Medford Water Commission customers receive Butte Spring water that typically does not have taste and odor concerns. The City of Grants Pass should be aware of the potential for taste and odor events in the future and will have to decide if investment in taste and odor control technology in the future will be beneficial to its customers. The City will also have to balance the need for taste and odor control with the risk of re-rating the plant if major process changes are made.

Trace Organics and Emerging Contaminants

Trace organics and contaminants of interest for the Rogue River supply which could become regulated within the next decade include:

- Hexavalent chromium
- Emerging contaminants
- Herbicides and pesticides
- Algal toxins

Concerns about the presence of hexavalent chromium have become elevated in the western United States, especially in California. Currently, only total chromium is regulated at an MCL of 0.1 mg/L (100 ppb). Hexavalent chromium is an identified carcinogen, but it is not currently known what a future MCL might be. It is not anticipated that hexavalent chromium will be a trace metal of concern in the Rogue River supply nor for the WTP.

The water industry's understanding of the treatment technologies needed to remove trace organics and emerging contaminants is in its infancy. These emerging contaminants include EDCs, PhACs and PCPs, all of which may be present in drinking water supplies, especially

those which receive discharges from wastewater treatment plants or stormwater runoff from urban and agricultural areas. Algal toxins are also an emerging trace contaminant of interest in surface water supplies. Most of these compounds are currently not regulated in drinking water, but it is possible that regulations will be promulgated in the future. Therefore, many drinking water providers are taking a close look at their treatment plant's ability to remove or destroy these compounds. Based on limited data searches, it does not appear that the Rogue River has been investigated for the presence of emerging contaminants.

Removal of Emerging Contaminants

Table 3-7 presents a summary of the anticipated performance of different types of drinking water treatment processes for removal of various classes of compounds based on the most recent industry research. Researchers have concluded that, in general, advanced treatment technologies such as activated carbon, high-pressure membrane processes (such as nanofiltration or reverse osmosis), and advanced oxidation (such as ozone or UV with hydrogen peroxide) are effective in the removal of many of these trace contaminants. However, no single treatment process has been demonstrated to be consistently effective in removing all of the emerging contaminants currently targeted due to the wide ranges in their physical and chemical properties.

It is anticipated that future drinking water treatment facilities will likely include one or more advanced treatment modules added to existing and new conventional treatment plants creating multi-barriers to a full range of potential existing and emerging contaminants. The existing Grants Pass WTP does not have any processes which can be considered excellent or good to reliably treat for emerging contaminants. Planning for emerging contaminants is addressed in subsequent chapters of this Facility Plan Update.

Historical Compliance

Grants Pass WTP staff began proactively monitoring for hexavalent chromium by testing samples monthly starting in February 2011, as suggested by EPA. As of March 2011, sampling has been reduced to quarterly per EPA recommendations. Figure 3-8 displays the results of this testing. Concentrations of hexavalent chromium are well below the total chromium MCL of 100 ppb. Chromium and hexavalent chromium are on the list for UCMR 3 testing.

Table 3-7
Unit Processes and Operations Used for Removal of Emerging Contaminants

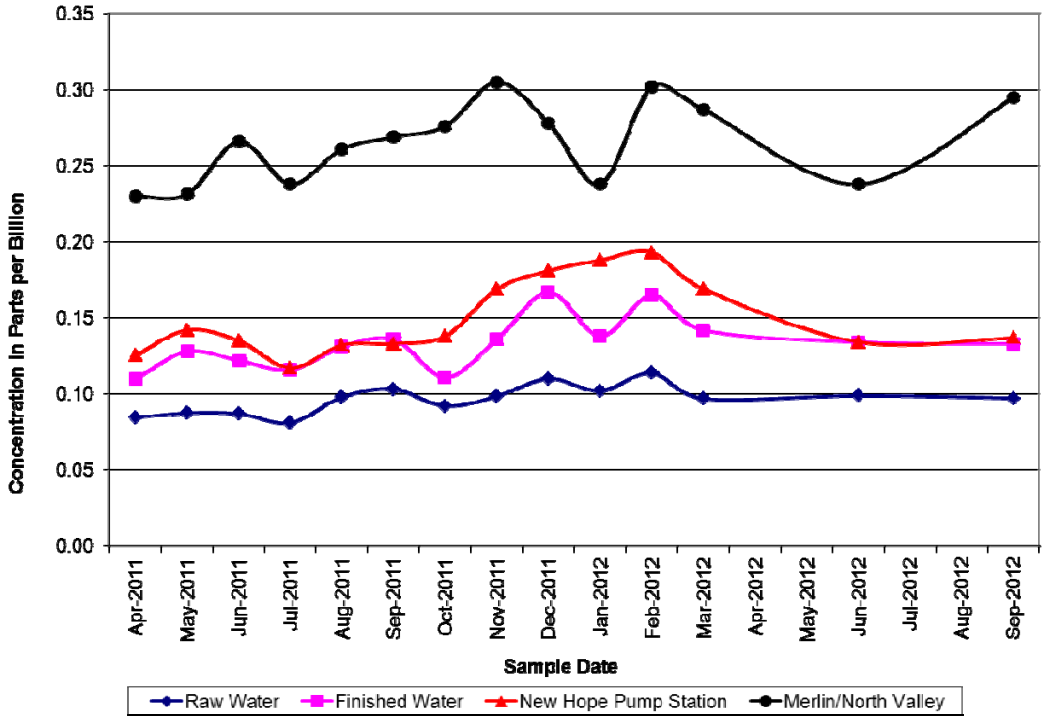
Group	Classification or Use	AC	BAC	O_3 and AOPs	UV and AOPs	CL_2 or ClO_2	Coagulation and Flocculation	Softening and Metal Oxides	NF	RO
EDCs	Pesticides	E	E	L-E	E	P-E	P	G	G	E
	Industrial chemicals	E	E	F-G	E	P	P-L	P-L	E	E
	Steroids	E	E	E	E	E	P	P-L	G	E
	Metals	G	G	P	P	P	F-G	F-G	G	E
	Inorganics	P-L	F	P	P	P	P	G	G	E
	Organometallics	G-E	G-E	L-E	F-G	P-F	P-L	P-L	G-E	E
PhACs	Antibiotics	F-G	E	L-E	F-G	P-G	P-L	P-L	E	E
	Antidepressants	G-E	G-E	L-E	F-G	P-F	P-L	P-L	G-E	E
	Anti-inflammatory	E	G-E	E	E	P-F	P	P-L	G-E	E
	Lipid regulators	E	E	E	F-G	P-F	P	P-L	G-E	E
	X-ray contract media	G-E	G-E	L-E	F-G	P-F	P-L	P-L	G-E	E
	Psychiatric control	G-E	G-E	L-E	F-G	P-F	P-L	P-L	G-E	E
PCPs	Synthetic musks	G-E	G-E	L-E	E	P-F	P-L	P-L	G-E	E
	Sunscreens	G-E	G-E	L-E	F-G	P-F	P-L	P-L	G-E	E
	Antimicrobials	G-E	G-E	L-E	F-G	P-F	P-L	P-L	G-E	E
	Surfactants and detergents	E	E	F-G	F-G	P	P-L	P-L	E	E

E: excellent (> 90%); G: good (70 – 90%); F: fair (40 - 70%); L: low (20 - 40%); P: poor (< 20%). Date and Source: Snyder et. al., 2003

Table Abbreviations

AC – Activated Carbon
EDCs – Endocrine Disruptors
O_3 – Ozone
AOPs – Advanced Oxidation Process
PCPs – Personal Care Products
RO – Reverse Osmosis
BAC – Biologically Activated Carbon
PhACs – Pharmaceuticals
UV – Ultraviolet Light
Cl_2 – Free chlorine
NF – Nanofiltration

Figure 3-8
Hexavalent Chromium Levels for the Grants Pass Water Treatment Plant and Distribution System



Summary

The Grants Pass WTP has consistently met all existing primary water quality regulations for over a decade. There are no major regulatory issues of concern at this time. However, there are some regulatory and water quality issues which the City should consider as part of future plant expansions and improvements:

1. Ensure that the plant continues to be rated as "complete conventional filtration," or its equivalent, to minimize the *Giardia* inactivation requirements.
2. Consider that potential challenges will arise if OHA decides to strictly enforce the post-filtration CT requirements (i.e., to achieve a minimum 0.5-log *Giardia* inactivation in the clearwell at all times).
3. Focus on treatment strategies and optimized plant and distribution system operations to minimize formation of DBPs.
4. Focus on producing a consistent finished water pH and alkalinity to continue complying with the Lead and Copper Rule.
5. Consider treatment process alternatives to reduce or eliminate earthy and musty tastes and odors which may possibly occur in the lower Rogue River during summer and fall based on what currently occurs in Medford.
6. Consider treatment process alternatives that can remove trace organics and emerging contaminants which may be present in the Rogue River or become a regulatory requirement in the future.

The biggest impacts to the plant processes, facility layouts, space requirements, and costs would come from regulatory changes by OHA related to disinfection compliance, complying with the Stage 2 D/DBP Rule, the City's decisions to implement taste and odor control, and control of emerging contaminants. These issues are discussed further in subsequent chapters of this Facility Plan Update.

CHAPTER 4
CAPACITY REVIEW

Introduction

This chapter presents a review of the hydraulic capacity and treatment process capacity of the existing WTP. This work will determine the current and possible future capacity of the WTP given the limitations of each process and the system as a whole. The hydraulic capacity is determined by the piping, pumping, and flow control systems. Each process or support system has its own capacity relative to certain design criteria or operating parameters which are independent of other unit processes. Presented as part of this work will be a determination of the most limiting or controlling process or feature of the WTP's capacity. As part of this capacity analysis, an estimation of the WTP's firm capacity will be made.

Hydraulic Capacity Evaluation

This section presents a methodology overview of the hydraulic capacity evaluation and results of this evaluation. The hydraulic capacity analysis performed in the previous 2004 Facility Plan used hand calculations to establish maximum and firm capacities of individual portions of the WTP from the intake to the finished water pumps. The analysis performed as part of this Facility Plan Update uses a computer model to simulate the hydraulic performance and plant operations and determine the impacts of specific existing limitations on upstream and downstream facilities that are hydraulically linked. Previous planning work used hand calculations for this determination of the plant's hydraulic capacity. The mathematical formulas used for open and closed conduit calculations are the same as those used for the 2004 plan.

Typical Plant Operation

This section describes standard operating procedures and physical conditions which were incorporated into the analysis.

Raw Water Pumping

Raw water pumps are operated at one of ten internally approved plant production or flow rates based on anticipated system demand. The approved flow rates have been developed to aid in water quality measurement and production calculations that are recorded for regulatory compliance monitoring. The approved flow rates are shown in Table 4-1.

Sedimentation Basins

Flow from the raw water pumps is split between sedimentation basins 1, 2 and 3. Flow is controlled and proportioned by throttling the inlet valve to basin 3. Further adjustment can be made through the positions of the mud valves at the inlets of basins 1 and 2. The plant is

typically operated with the inlet valve to basin 3 throttled to control flow split and optimize individual basin resident time for compliance purposes. Basin dimensions and flow splitting are summarized in Table 4-2.

Table 4-1
Approved Plant Flow Rates

	Plant Production					
Gallons per minute	3,560	4,500	5,500	6,500	7,300	8,500
Million gallons per day	5.1	6.5	7.9	9.4	10.5	12.2
Gallons per minute	9,500	10,500	11,500	12,500	13,500	13,900
Million gallons per day	13.7	15.1	16.6	18.0	19.4	20.0

Table 4-2
Sedimentation Basin Summary

Parameter	Unit	Basin 1	Basin 2	Basin 3
Width	ft	61	38	80
Length	ft	98	98	80
Depth	ft	13	13	13
Surface Area	ft^2	5,978	3,724	6,400
Volume	ft^3	77,714	48,412	83,200
	gal	581,301	362,122	622,336
Tank Flow Percent of Total	%	37	23	40

Filtration

Water from each basin flows by gravity over weirs within the basins and collected by launders. The water flows from the launders and fills a common channel which conveys flow to the filters. The common channel water level is monitored at three points: near filters 3 and 5 and between filters 7 and 8. The water level in the common channel is kept at 3.6 feet (the total height of the channel is approximately 5 feet) by the plant's control system, which adjusts flow to each group of filters. The flow is optimally split by a ratio of flow to area and the number of filters in service. Filter dimensions and flow splitting are summarized in Table 4-3.

Flow through each filter is controlled by throttling the effluent valve on the filter. As the head loss through the filter increases due to increased flow or solids loading, the filter's effluent valve is opened farther to maintain a constant flow. Backwashes are initiated when the head loss through the filter is greater than 7 feet, turbidity is greater than 0.15 NTU, or the filter has not been backwashed for 80 hours. The maximum backwash time criteria includes both time in operation and time offline.

Table 4-3
Filter Summary

Parameter	Unit	Filters 1 to 3	Filters 4 and 5	Filters 6 to 8
Length	ft	17	21	18
Width	ft	15	18	18
Area of Each Filter	ft^2	255	378	324
Total Area	ft^2	765	756	972
Filter Flow Percent of Total	%	31	30	39
Each Filter Flow Ratio	%	10	15	13

The plant typically operates with one filter offline. The offline filter is brought online when another filter needs to be backwashed. This control strategy has helped to eliminate surges in the filter levels and a corresponding fluctuation in plant flow rate. At flows greater than 15 mgd, or if water is backing up into the common inlet channel (which can occur during maintenance activities that leave facilities offline), operation of all filters becomes necessary. Filter effluent is collected in closed manifold piping and flows to the clearwell.

Finished Water Storage

The clearwell is operated at a fixed water level that maximizes chlorine contact time for regulatory compliance. As the level rises and falls, the effluent pumps increase or decrease their speeds to maintain the constant water level, currently set at 14.5 feet.

Hydraulic Model

A digital hydraulic model was developed to determine the hydraulic capacity of the various conveyance systems at the WTP. The following sections describe model development, input, and results. Visual Hydraulics, a commercially available hydraulic analysis software program, was used to develop and run flow scenarios to assess the plant's hydraulic performance and to identify areas of hydraulic concern. These areas of concern were then further analyzed using hand calculations and discussion with City staff.

Conveyance Systems

The Visual Hydraulics program analyzes water surface profiles of water conveyance systems. Specifically, a downstream control point is selected, and the hydraulic profile is then determined upstream of that control point. Review of historical WTP record drawings were used to initially develop the model. Table 4-4 summarizes the values used in the model for different criteria. See Appendix B for a Hydraulic Model Schematic.

Table 4-4
Hydraulic Parameters Summary

Condition	Equation	Parameter	Value
Pressure Pipe	Hazen-Williams	C-coefficient	110
Pressure Pipe	90-Degree Bend Minor Loss	K-value	0.25
Pressure Pipe	Entrance Minor Loss	K-value	0.5
Pressure Pipe	Exit Minor Loss	K-value	1.0
Open Channel	Manning's Equation	Manning's *n*	0.013

Revisions were made to the model using City input and iterative refinements. The following is a summary of changes incorporated in developing the final hydraulic model.

- Minor Losses – Typical design values for minor losses, e.g. pipe entrance and exit losses, were used where applicable.
- City Experience – Through conversations with City staff, input on the hydraulic performance of the WTP was collected and compared to the preliminary results of this study. For example, filters 4 and 5 are not able to handle as much flow as would be anticipated from splitting flows based on comparative surface areas within the WTP.
- Flow Split – The flow split to the sedimentation basins and the filters was modified to more evenly match head loss through a basin or filter train relative to the other basins or filters.

Failure Criteria

In estimating the maximum hydraulic flow through the WTP, the flow used in the hydraulic model was increased in 0.5-mgd increments until one or more of the failure criteria were met. The failure criteria are as follows:

- Loss of containment – The estimated water levels across the entire treatment plant were compared to the top of the holding structures to determine if the plant flow being modeled would be contained within the system.
- Weirs fully flooded – The flow being modeled was considered to be at failure once a weir had become fully submerged and no appreciable drop was predicted across the weir.
- Adverse impact water elevations – The final failure criteria involved determining if the predicted water level would have an adverse impact on the operation of mechanical equipment at the plant.

The acceptable flow for the various flow scenarios was assumed to be 0.5 mgd below the flow triggering failure.

Pump Station Conveyance

There are three pumping facilities at the WTP: raw water pumping, high service or finished water pumping, and backwash pumping. The initial capacity rating of each facility was based on equipment data, supplemental information provided by the City, and previous documentation. The capacity of each facility was determined for two conditions, total capacity and firm capacity. Total capacity is the production capacity with all pumps in operation. Firm capacity is the production capacity with the largest pump out of service. This section includes descriptions of each pumping facility and their associated total and firm capacity assessments.

Raw Water Pumping

The WTP uses four 75-HP vertical turbine pumps for raw water pumping. Each pump has a design capacity of 3,200 gpm at a design TDH of 65 feet. They are each Worthington model 15HH-340 pumps. Since the 2004 Facility Plan, VFDs have been added to pumps 1 and 4 to allow additional flexibility in producing desired flow rates and splitting operational hours between pumps. The pumps were installed in the early 1980s when the raw water intake was built. Based on the design point, the pump station has a total capacity of approximately 20.2 mgd and a firm capacity of approximately 15.15 mgd.

There is space for six pumps within the pump station and if similar pumps are installed, the total pumping capacity would theoretically be 30.2 mgd. Based on comparing testing of flow and pressure in the raw water discharge line and head loss calculations, the raw water pumps may have been oversized, i.e., the design TDH is more than actual TDH. The pumps are likely pumping at higher flows than the original design anticipated, and a design-level analysis is needed to more accurately determine the actual capacity increase.

High Service Pumping

There are six pumps that transfer finished water from the clearwell into the distribution system. The size, design capacity, and pump control scheme is summarized in Table 4-5. The pumps are controlled by the staff and the SCADA system based on the distribution system demand. They are also operated to maintain the water level in the clearwell necessary to meet chlorine contact time (CT) requirements. Based on design points, the total pumping capacity is approximately 29.7 mgd, with a firm capacity of approximately 23.9 mgd. Assuming the velocity in the 36-inch diameter finished water pipeline is limited to a velocity of 6 feet per second (fps), the capacity of the pipeline that the pumps discharge to is approximately 27.4 mgd. Using a velocity of 6.5 fps, the existing 36-inch diameter pipeline is capable of conveying approximately 30 mgd. With the existing surge tank and the addition of three VFDs, the potential for surge has been reduced for these pumping facilities.

Table 4-5
High Service Pumping Summary

Pump Number	Model	Size (HP)	Head (ft)	Flow (gpm)	Control
1	Worthington Model 15HH-340	250	210	3,500	Soft Start
2	Fairbanks Morse Model 18HC	300	210	4,000	On/Off
3	National pump Company/Worthington Model H14XHC	250	220	3,500	VFD
3A	National pump Company/Worthington Model H14XHC	250	220	3,500	VFD
4	Worthington Model 15HH-340	250	210	3,500	On/Off
5	Worthington Model 15HH-277	200	210	2,600	VFD

Backwash Pumping

The two backwash pumps, including one pump that has just recently been added, are vertical turbine pumps which pump water out of the clearwell. Both pumps are controlled by VFDs. Table 4-6 summarizes the backwash pump capacity. The station has a redundant pump if one pump is not operable due to maintenance or damage. Because both pumps are on VFD control, the backwash system is able to prevent excessive surges in the backwash system and limit the flow velocity in the discharge line.

Table 4-6
Backwash Pumping Summary

Pump Number	Model	Size (HP)	Head (ft)	Flow (gpm)
1	Peabody Floway 22-BLK	200	62	7,000
2	Goulds Water Technology VIT-FFFM	150	60	7,600

Hydraulic Capacity Analysis Results

The following sections summarize results of the analysis and improvements that could increase hydraulic capacity.

Model Results

The hydraulic model of the WTP was first used to simulate plant operations as described in this chapter. At a plant flow of 21.0 mgd, the sedimentation basin weirs became flooded. If these weirs become flooded, flow splits in the plant will become more difficult to control and the sediment and floc loading to the filters will increase, diminishing their performance. However, this condition is not considered a failure for the WTP overall because the plant can still operate hydraulically above this flow. At a flow over 23.0 mgd, the mixing basin before

sedimentation basins 1 and 2 is flooded and loses containment. At 23.0 mgd, the WTP could no longer pass additional flow, and this is considered the maximum hydraulic capacity of the WTP. Table 4-7 shows a summary of the hydraulic profile of the plant at 23 mgd.

Table 4-7
Hydraulic Summary at 23 mgd Maximum Capacity

<table>
<tr><th colspan="2">Hydraulic Element</th><th colspan="3">Water Surface Elevation</th><th rowspan="2">Limiting Criteria</th></tr>
<tr><th>Downstream</th><th>Upstream</th><th colspan="2">Downstream</th><th colspan="2">Upstream</th></tr>
<tr><td>Distribution System</td><td>Clearwell</td><td colspan="2">1,085 to 1,108 (70 to 80 psi)</td><td colspan="2">922.96</td><td>Pipe Velocity</td></tr>
<tr><td>Clearwell</td><td>Common Filter Channel</td><td colspan="2">922.96</td><td colspan="2">935.34</td><td>Pipe Velocity, Head Loss Through Filter, Loss of Containment</td></tr>
<tr><td rowspan="3">Common Filter Channel</td><td rowspan="3">Sedimentation Basins</td><td colspan="2" rowspan="3">935.34</td><td>Basin 1</td><td>935.39</td><td rowspan="3">Loss of Containment</td></tr>
<tr><td>Basin 2</td><td>935.42</td></tr>
<tr><td>Basin 3</td><td>935.49</td></tr>
<tr><td rowspan="2">Sedimentation Basin</td><td rowspan="2">Mixing Basin</td><td>No. 1</td><td>935.39</td><td colspan="2" rowspan="2">936.00</td><td rowspan="2">Weir Submergence, Loss of Containment</td></tr>
<tr><td>No. 2</td><td>935.42</td></tr>
<tr><td>Mixing Basin</td><td>River Intake</td><td colspan="2">936.00</td><td colspan="2">886.00</td><td>Pipe Velocity</td></tr>
</table>

After the initial results were obtained, operating parameters were changed in the model in an effort to determine if higher flows could be passed by the WTP. Optimization included splitting flow to the filters and basins in a manner that more evenly matched head loss through a train. Under this analysis, at a flow over 23.0 mgd, the sedimentation basin weirs are flooded. At flows over 24.5 mgd, the mixing basin is flooded. Operating the WTP in a manner similar to the optimized model would entail iterative adjustment of both manual and automated valves that control individual contact and filter basins and would result in differing contact times that would make regulatory compliance difficult to achieve. For this reason, the higher flows are not considered practical.

Maximum and Firm Hydraulic Capacities

Based on design capacity alone, the WTP capacity is currently limited by the raw water pump station capacity. The maximum overall hydraulic plant capacity is 20.2 mgd. The firm hydraulic capacity, with the largest river intake pump out of service, is approximately 15.1 mgd.

Increasing Hydraulic Capacity

Using the projected water demands, the current maximum plant capacity of 20.2 mgd will meet projected system MDD until the year 2028. Improvements could be made to the WTP to increase its hydraulic capacity to 25 mgd. The WTP would then be able to meet projected system MDD until year 2046. Increasing the hydraulic capacity of the WTP to 25 mgd

would require substantial capital investments in the form of additional basin and conduit upgrades. If implemented collectively, the following improvements would increase the maximum plant hydraulic capacity to 25 mgd:

- Increase river intake pumping capacity by installing additional pumps or modifying existing pumps.
- Enlarge submerged opening in mixing basin baffle wall.
- Add additional launders to basins 1, 2, and 3.
- Filters 1 through 5 effluent pipe gallery modifications including weir plate invert set at consistent 926.62 feet.
- Perform operational tests of the raw water pump station and the high service pump station to determine the firm capacity of these facilities under actual operating conditions.

Process Capacity Evaluations

The capacity of each of the plant processes was evaluated for its ability to meet existing production needs and to estimate its maximum capacity. The evaluations are summarized in this section.

Chemical Feed Systems

The WTP's primary chemical storage, metering, and feed systems at the plant include:

- Liquid alum (50 percent) for coagulation
- ACH for coagulation
- Liquid sodium hypochlorite (12.5 percent) for disinfection, pre- and post-chlorination
- Dry polymer for filter aid
- Dry $KMnO_4$ for taste and odor control, used intermittently

The first four systems are typically used continuously whenever the plant is in operation. Potassium permanganate is used only during infrequent taste and odor events. The doses of each chemical depend on the plant production rate and raw water quality.

Alum

Alum is stored in a 6,000-gallon fiberglass tank inside the WTP's chemical storage room. Alum is added to the raw water to aid in coagulation prior to static mixing. Alum is dosed using positive displacement diaphragm pumps. The pumps are rated at 39.6 gph at 58 psi, and the other is rated at 15.9 gph at 145 psi. These pumps are also used to feed ACH (see below).

When alum was the only coagulant used, the maximum day alum usage from 2004 to 2007 was 2,445 ppd. With the use of ACH, the maximum day usage for alum was reduced to 1,123 ppd. The corresponding maximum day ACH usage rate is approximately 690 ppd. Both maximum coagulant usage days had similar water quality and flow parameters.

At the current maximum instantaneous plant flow of 20 mgd, an estimated maximum alum usage rate is 1,250 ppd at an alum dose of 7.5 mg/L. This equates to a maximum chemical pumping rate of 4.9 gph using 5.4 pounds of alum per gallon of solution, which is less than the current rated pumping capacity of the alum feed pumps. It is not expected that chemical feed pumps would need to be replaced due to increased demand requirements.

Chemical storage quantities depend on a plant's proximity to chemical distributors and ability to have chemicals delivered at any time of the year. It is typical to maintain 15 to 30 days of chemical storage based on maximum dosage and ADD. The current ADD is 5.5 mgd and the current alum dose is 25 mg/L. For these flow conditions, the necessary alum storage is approximately 3,250 gallons for 15 days or 6,500 gallons for 30 days. This is more than the 6,000-gallon tank storage at the WTP. There are several suppliers of alum nearby and the ACH dosage could be increased and alum dosage decreased, so this slight lack of alum storage volume does not appear to be of immediate concern.

Aluminum Chlorohydrate

Aluminum chlorohydrate is stored inside the WTP's chemical room in a 6,000-gallon fiberglass tank which was formerly used to store alum. ACH is added to the raw water at the static mixer with alum. ACH is dosed using positive displacement diaphragm pumps. Since beginning use of ACH on a daily basis, the average dose was 15.8 mg/L and the plant used an average of 54.4 gpd. Under average conditions, the plant has more than 100 days of storage of ACH using the 6,000-gallon tank.

Sodium Hypochlorite

Liquid sodium hypochlorite is delivered and stored at the plant in three fiberglass reinforced plastic tanks, each with a capacity of 2,300 gallons, for a total storage capacity of 6,900 gallons. These tanks are located inside the hypochlorite feed room adjacent to the chemical feed room. The storage tanks and metering pumps are located within a concrete containment area to contain a major leak. There are three positive displacement mechanical diaphragm metering pumps, each rated at 24.0 gph. Under normal operating conditions, one pump is dedicated for pre-disinfection, injecting into the static mixing vault. Another pump is for post-disinfection with injection into the clearwell. The last pump serves as backup. Space and a piping connection have been included for a future pump.

At the current maximum instantaneous plant flow of 20 mgd, the estimated hypochlorite usage is 500 ppd at a combined pre- and post-chlorination dose of 3.0 mg/L. The dosage used in this calculation conservatively estimates hypochlorite usage during peak season demands. This equates to a total chemical pumping rate of 20.9 gph total, or 10.5 gph per

pump, well below the 24.0 gph rating of the current feed pumps. Using this same dose at 30 mgd, the existing pumping system should be capable of reliably meeting plant demands.

At the current ADD of 5.5 mgd and a maximum hypochlorite dose of 3.0 mg/L, hypochlorite storage required is approximately 2,000 gallons for 15 days and 4,000 gallons for 30 days. During periods of low demands, some utilities dilute the chemical to a concentration of 10 percent or less to reduce degradation of the chemical associated with longer holding times. Existing on-site storage capacity is sufficient for peak demand flows in excess of 30 mgd, providing more than 15 days of storage. No additional hypochlorite storage will be required in the foreseeable future.

Polymer

A low-molecular-weight polymer is added to the filter influent pipelines as a filter aid to improve filter performance. A dry feed system, including two 290-gallon mix/aging and feed tanks and one diaphragm positive displacement metering pump rated at 15.9 gph (at 145 psi), are used to make and feed the solution. Eight rotameters split the feed to each filter's influent pipe. Dry polymer is shipped in 55-pound bags and stored adjacent to the mixing tanks in the chemical room.

Using a filter aid dose of 0.05 mg/L and a plant flow of 20 mgd, the polymer used would be approximately 8.3 ppd. At 30 mgd, the plant would use approximately 12.5 ppd. The existing system is adequately sized and improvements or upgrades are not anticipated for increased demand. If improvements are made to the clarification process, the filter aid requirements and dosages would most likely decrease.

Potassium Permanganate

The plant infrequently adds potassium permanganate to the raw water pipeline and mixing basin for taste and odor control. The permanganate feed pump is a volumetric pump ($^1/_3$ HP, 1,800 rpm) type with a hopper that discharges to a flushing funnel and eductor which discharges the resulting solution to the application point. Prior to injection, the permanganate solution is further diluted; dilution water is controlled by a solenoid valve. Dry potassium permanganate is shipped in 110-pound steel drums and stored between the permanganate feeder and the polymer metering pumps.

Assuming a dose of 0.25 mg/L and a plant flow of 20 mgd, the permanganate used would be approximately 41.7 ppd. At 30 mgd, the plant would use approximately 62.5 ppd. The existing system is adequately sized and would not be expected to need improvements or upgrades for increased demand.

Coagulation Performance

Water from the Rogue River is generally considered a low turbidity, good quality supply, but some treatment challenges exist due to seasonal and diurnal variation in pH, seasonally

variable turbidity, temperature, and occasional taste and odor events. These variable raw water quality conditions can significantly impact coagulation and sedimentation performance at the plant.

Elevated turbidity was historically treated using high doses of alum. High doses of alum corresponded with increased solids production and, in turn, put high stress on the old solids handling facilities. Increased alum also depressed pH to levels where pH adjustment chemical was required to bring the pH back to targeted levels for corrosion control. This resulted in higher overall chemical and operations and maintenance costs with a reduction in plant efficiency.

After the 2004 WTPFP, the plant began experimenting with different alternative coagulation chemicals and now uses two coagulants at the plant. PASS-C, a PACl derivative, was originally used until the plant transitioned to an ACH derivative. Alum usage and overall coagulant usage has decreased significantly under current operations and there is no longer a need for a pH adjustment chemical at the plant. The original lime feed system has already been decommissioned and removed from the plant.

Sedimentation Basins

The sedimentation basins currently provide contact time for disinfection and some solids removal prior to filtration; no formal flocculation is provided in the basins other than mild hydraulic turbulence. Basins 1 and 2 have a combined rated capacity of 12 mgd; basin 3 is rated at 8 mgd , so the total rated process capacity is 20 mgd. The basins provide satisfactory water for filtration most of the year. However, all basins experience challenges with regard to short-circuiting, high solids loading to the filters, sub-optimal flocculation and seasonal turbidity spikes. Basin 3 is particularly vulnerable to short-circuiting. In addition, there is no continuous solids removal system; as solids accumulate in the basins, effective volume is reduced, compromising CT compliance and reducing settling efficiencies.

Selected design criteria for the existing basins were summarized and compared to criteria that are considered optimal for pretreatment in the 2004 WTPFP. Based on the comparison, several improvements to the basins which could be made to ensure the current plant capacity can be fully realized are:

- Incorporation of formal flocculation by either mechanical or hydraulic means for improved settled water quality
- Installation of a continuous residual solids removal system to minimize short-circuiting associated with solids accumulation and to equalize residual solids loading to the solids handling system
- Installation of internal baffling in basin 3, in addition to flocculation, to minimize short-circuiting resulting from the geometry of the basin

The City completed pre-design of automated residual solids removal for basins 1 and 2 in February 2010, which also reviewed flocculation alternatives, but the project was deferred due to high costs which included significant structural improvements.

The suggested improvements are intended to optimize the treatment process, but will not necessarily increase the process capacity of the basins. Alternatives to address these process limitations are discussed in detail in Chapter 7.

Filtration

Chapter 2 presents a detailed evaluation of historical filter performance and a discussion of possible capacity limitations. The filter improvements made in 2006 have significantly improved filter performance. However, there are some deficiencies identified as part of the historical performance analysis and filter investigations which include the following:

- Filter production efficiencies currently range from 90 to 94 percent; 97 percent is considered the minimum desirable filter production efficiency.
- Plant records show that filters 6, 7 and 8 are backwashed approximately 25 percent more frequently than the other five filters. This can be attributed to short-circuiting of water through basin 3 (more turbid settled water than from basins 1 and 2) and therefore higher solids loading rates to the filters which increases the head loss accumulation rate. Flow-splitting or other improvements made to clarification may help balance filter run times to increase overall plant efficiency.
- The existing surface wash system is not optimal and regular cleaning by hand is required. The addition of an air scour system and filter trough modifications could help improve cleaning and reduce overall operations and maintenance.

Certain deficiencies in the sedimentation basins and filter media design make it difficult to operate the plant at 20 mgd for extended periods without frequent filter backwashes. This is consistent with plant operations staff experience. The existing filters are not adequate for flows higher than 20 mgd, so modifications to the filters or additional filters would be required to increase capacity. A discussion of alternatives to address these issues is presented in Chapter 7.

Clearwell

The existing 433,000-gallon clearwell is relatively small for a 20-mgd plant; CT compliance at the plant is only possible by carefully monitoring and controlling the chlorine residual through the basins, and also by not exceeding certain operating flow rates during winter and spring due to water quality constraints. The use of VFDs on selected high service pumps helps maintain a relatively high water level in the clearwell. However, multiple “back-to-back” backwashes can create challenges to CT compliance because this tends to lower the clearwell level. Running the plant at lower production rates for longer periods of time during

challenging water quality conditions, mainly cold water events, can help ensure continued CT compliance in the near-term.

Clearwell volume will need to be expanded in the future when plant demands exceed 20 mgd if free chlorine continues to be used for primary disinfection. Alternatives to integrate additional clearwell volume with the existing clearwell and high service pump station are discussed in Chapter 7.

Disinfection and Disinfection Byproduct Formation

The plant is currently capable of meeting CT requirements within the existing basins and clearwell by using pre-chlorination residual and maximizing the operating level in the clearwell. However, the dependence of disinfection compliance on the contact time achieved through the basins significantly limits operational flexibility at the plant; free chlorine residual must be carefully monitored and maintained through the basins to meet CT requirements. In addition, efforts to increase the pre- and post-chlorination residual must be balanced with disinfection byproduct (DBP) control. Process challenges in meeting CT are related primarily to increased demands during the spring and fall when demands are still fairly high and water temperatures are lower. Chapter 3 discusses this issue and how the plant could make operational adjustments to run the plant for longer periods during these times to still meet CT.

Disinfection and DBP regulations may drive disinfection improvements at the plant in the coming years if ongoing monitoring indicates elevated concentrations of these compounds within the distribution system. Alternate process modifications may be necessary to avoid the reliance on free chlorine for disinfection. Such processes may include ozone or UV irradiation. Discussions of improvement alternatives for each case are presented in Chapter 7.

Washwater and Solids Handling Systems

The 2004 WTPFP concluded that the old mill pond was full of residual solids and needed to be cleaned. The old mill pond was deemed to be inadequate for residual solids drying and an alternative method for solids handling was needed. The City determined that mechanical dewatering systems were cost-prohibitive and that solar drying lagoons were space-prohibitive. The plant transitioned to an approach that utilizes geofabric bags for dewatering solids.

Residual solids conditioned with dewatering polymer are loaded into the geofabric bags and allowed to drain and dry. Once the dewatered residual solids are considered dry enough, the bags are cut open and the dried solids are hauled off-site for disposal. There is space reserved on the plant site for dewatering the residual solids from the sedimentation basins. The old mill pond is dredged using a remotely operated dredging system to bring residual solids to shore to be placed into the geofabric bags for dewatering on-shore. This current practice is effective and requires little maintenance, but it is labor-intensive and requires a lot

of space. As plant production increases and space is needed for expansion or plant upgrades, an alternate solids handling approach will be necessary. A detailed discussion of alternative solids handling and disposal methods is presented in Chapter 6.

Summary

A summary of findings from the hydraulic capacity and treatment process evaluations is presented below. Alternatives to address deficiencies at the existing WTP are presented in Chapter 7.

- The existing raw water pumps and finished water pumps are capable of pumping at least 20 mgd into and out of the plant.
- The firm hydraulic capacity of the plant is approximately 15 mgd. Installation of an additional 5 mgd of raw water pumping capacity would provide added operational flexibility and redundancy when plant demands reach 15 mgd, which is anticipated to occur within the next 5 to 10 years.
- The current maximum hydraulic capacity of the plant is 21 mgd. Significant modifications and improvements would be required to provide more hydraulic capacity in the existing plant.
- The chemical systems appear to be adequate to meet demands for the next 10 years except for periodic maintenance and replacement. This equipment may need to be supplemented to provide additional capacity or replaced if the plant capacity is expanded beyond 20 mgd.
- The existing sedimentation basins have a maximum process capacity of 20 mgd. Additional clarification capacity is required if the plant is to be expanded. Also, basin 3 is not as efficient as basins 1 and 2 due to the square geometry and radial flow pattern. This deficiency inhibits filter and plant performance at higher flows.
- The existing filters have a maximum process capacity of 20 mgd. Additional filters are required if the plant capacity is to be expanded.
- Continuous residual solids removal systems in the sedimentation basins would equalize solids loading to the solids handling system, maximize the chlorine contact time and settling time by minimizing solids accumulation, and eliminate the need for taking basins out of service for cleaning. Basins cannot currently be taken out of service for solids removal during the summer months and this can become a constraint in the future as water demands and solids production increase.
- The plant is currently capable of meeting CT requirements as long as flow is restricted to 10 mgd during winter and spring. The clearwell will need to be expanded as plant demands increase or another method of disinfection will be required.
- The strategy of dredging the old mill pond on a semi-regular basis and periodically removing solids from the sedimentation basins is effective, but is labor- and time-intensive. As plant demands and solids production increase, the plant site may no longer be able to process all of the solids. An alternative long-

term strategy for solids handling and disposal will be necessary if the existing plant will continue to be used for the next 10 to 20 years, or longer.

CHAPTER 5
EVALUATION OF EXISTING WATER TREATMENT PLANT FACILITIES

Introduction

In this chapter, each of the existing plant's systems and structures is reviewed to determine if improvements are required, and to estimate remaining useful life. The results of this review are integrated with the regulatory and capacity reviews to develop capital improvement recommendations to maintain existing capacity and to increase capacity, if needed. This chapter also presents a review of key structural and seismic risks and recent structural testing results that provide additional information related to this critical aspect of the existing plant. As mentioned in Chapter 1, one of the key drivers for completing this WTPFP update is concern about continuing to invest in the existing plant given its age and structural vulnerability.

Structural and Seismic Risks

Over the past few years, the WTP staff has observed cracks and other observable damage to the concrete when making minor repairs in basin 1. This damage has presented in the form of concrete loss, staining from reinforcing steel or reinforcing steel wire spacers, oxidation, and rusting. Some visual deficiencies were noted consisting of some minor hairline cracking and softness at the top of the walls where the concrete is most exposed to the weather, and where a freeze-thaw environment exists.

Structural Review

A seismic and structural review of the Grants Pass WTP was completed in 2011. A review of geotechnical studies previously conducted at the plant site show that ground-shaking and slope stability along the Rogue River bluff are the two most significant seismic geotechnical risks. A review of the construction documents of the plant shows that, overall, the structures appear to have been designed and constructed prior to consideration of seismic loads. Due to the lack of seismic design, or inadequate design for seismic loads, the facility is judged to have a high seismic risk with portions susceptible to collapse in a strong earthquake. In general, major structural elements and connections of the lateral-load carrying systems were not designed to conform to current code requirements. They were also not designed for expected wind load performance for this type of structure. A planning-level project cost to address the various structural deficiencies observed during the review was estimated to be approximately $8.5 million. This review is documented and on file with the City's records. A technical memorandum summarizing the structural review of the clearwell is included with this report as Appendix C.

A seismic event could cause complete loss of water supply for the City. A severe wind event could cause a prolonged outage. With no apparent second source of supply available to the City, these risks are unacceptable. As such, all capital improvement alternatives developed

to upgrade the existing WTP include the recommended structural upgrades as a baseline requirement.

Carbonation Testing

After initial discussions with City staff, different types of concrete and structural testing methods were reviewed to further define areas in the plant which may be subject to concrete or rebar failure within the current planning horizon. The goal of this physical assessment was to provide further information regarding the remaining useful life of the existing facilities and to better inform the decision making process.

To further investigate the condition of critical structures where cracking and leaking has been observed, the City conducted carbonation tests. Carbonation testing measures pH to determine the degree of degradation of the concrete. These tests were conducted in parallel with semi-annual basin cleaning in late October 2012.

The assessed condition of the basins is based on observations, soundings, and carbonation testing. The exterior concrete of the basins is in good condition. The interior concrete of the drained and accessible basins 1, 2, and 3 is in good condition. There are, however, some areas of concern.

In basin 1, the walls were immediately noted as having internal areas with some through-thickness cracking and some surficial concrete spalling of a depth less than ½ inch and varying lengths and widths in both directions. It appears that the strength of the concrete around the spalled areas has been deteriorating. Some minor cracks on the perimeter walls were observed that had efflorescence present for a long period of time and had no water seepage. The efflorescence indicates autogenous healing of the concrete had already taken place on these minor cracks. No serious rusting of rebar on the external surfaces of the walls was observed. Previous inspections of the West and East clearwells found that the interior walls and ceiling of the original clearwells have multiple locations where rust is bleeding through to the inside face of the wall from multiple reinforcing steel locations, most notably below the original control room. This is an indication of rebar corrosion.

As observed by the WTP staff, the concrete and reinforcing steel in most elements of basins 1, 2, and 3 are in relatively good condition with only limited carbonation depths and rusting. The concrete pH is in the suitable range for protection of the reinforcing steel and hence the composite reinforced concrete system meets the criteria of being able to protect the reinforcing steel.

Test Results

Based on the results of the initial round of carbonation testing in basins 1, 2, and 3, it does not appear to be necessary to conduct additional concrete testing. From a non-seismic perspective, the current assessment of basins 1, 2, and 3 from the areas inspected and tested is that the concrete and reinforcing steel are in generally good condition with some wear after

up to 80 years of service. However, there is evidence of corrosion damage to concrete in the original clearwells.

Based on these tests and previous inspections and analyses of the existing plant structures, it is believed that the remaining useful life can be in excess of 40 years should the seismic and structural upgrades be completed.

Plant Inventory and Condition Assessments

The following is a discussion of each major system at the WTP, including pertinent information and observations used to determine remaining useful life and suggested capital improvements associated with the equipment. Table D-1 in Appendix D provides a detailed summary of plant equipment.

Raw Water Intake and Pump Station

The intake and pump station were constructed in the early 1980s as part of the last major plant expansion. The intake screening system was upgraded in 2008 and is equipped with four wedge wire screen panels that provide screening with a capacity of approximately 30 mgd. The intake screen also includes an articulating arm wash system. Accumulation of silt and sediment on the screens and in the wet well has become a problem since the Gold Ray Dam was removed in 2010. Divers have been employed to help remove sediment from the space between the screens and the base of the concrete structure. The City evaluated the possibility of implementing a low-cost fixed spray nozzle system to reduce sediment build-up at the base of the screen face in 2013. These improvements were deferred pending the completion of this Plan. If the existing WTP intake is going to continue to be used, improvements to address sediment accumulation will likely be required.

The four existing raw water pumps were installed in 1983 when the new intake facility was constructed. There is space available to add two more pumps. Since installation, the pumps are rebuilt and the pump impellers replaced on an approximate 6-year cycle. The pumps were originally water-lubricated, but were converted to oil lubrication after recurring problems were observed with the water-lubricating system. The pumps occasionally have had issues with taking oil after being rebuilt, but have generally worked well. With continued maintenance and repair, the pumps should have significant remaining useful life. As described in previous sections, the firm raw water pumping capacity is approximately 15 mgd. Installation of an additional pump is required when demands approach 15 mgd to reliably deliver a peak flow of 20 mgd.

The raw water pumps have performed well and are in no need of immediate attention. Recent VFDs installed by the City on two of the raw water pumps have provided better flow control of the plant and added operational flexibility. See the Photo Log, Appendix D – River Intake for additional details and photos.

Chemical Storage Areas

The plant has five chemical storage and feed systems, including:

- Liquid alum
- Liquid ACH
- Liquid sodium hypochlorite
- Dry polymer
- Dry potassium permanganate

A brief description of the plant's chemical storage areas is presented below.

Sodium Hypochlorite Room

Liquid sodium hypochlorite is stored in three cylindrical fiberglass reinforced plastic tanks with volumes of 2,300 gallons each. The tanks are housed in a room adjacent to the main chemical storage area. There is a containment system provided and it is ventilated by a three-fan system. The sodium hypochlorite solution has a 12.5 percent solution strength and is fairly corrosive. Plant staff has observed that joints in piping must be continuously monitored for leaks and have been replaced fairly frequently. Due to the size of the tanks and room configuration, only one tank, the tank adjacent to exterior building wall, can be removed for maintenance. If there is a problem with either of the other two tanks, they would need to be repaired in-place or would require the adjacent tanks to be moved out of the building in order to perform work on them. In the case that the tank most inward in the building experiences failure, the tank would have to be demolished in place and removed in pieces. In addition, the paint in the room has started to peel and may require maintenance to prevent corrosion to the interior of the building.

There are three positive displacement diaphragm sodium hypochlorite feed pumps also located in the sodium hypochlorite room. A limitation of these pumps is that there is no alarm system to alert plant staff of a pump failure. The only way the plant staff can ascertain chlorine feed pump failure has occurred is by observing a steady decrease of the chlorine residual in the clearwell. See the Photo Log, Appendix D – Sodium Hypochlorite Room for additional details and photos of the equipment.

Chemical Storage Room

The chemical storage room houses the rest of the chemicals used at the plant and their associated feed systems. Plant space is at such a premium that the main maintenance area for the plant is co-located in the chemical room along with the WTP's pilot filters and the portable pilot filter used for public outreach and plant operation demonstrations. Removal of the lime equipment has provided some limited additional space for storage.

In general, all chemical feed systems have been maintained very well, are in good condition, and should reliably meet the City's needs for many years. However, chemical feed equipment has a finite useful life and will likely need to be replaced at least a few times during the planning horizon considered for this report. The replacement schedule will depend on when the equipment was installed and will vary. The City should also consider chemical feed system replacements if the plant capacity at the existing site is expanded. This is recommended because as feed pumps age, it can become more difficult to find replacement parts and perform maintenance, especially if models are discontinued or a manufacturer goes out of business or is purchased by another company that no longer offers service.

Following a recommendation in the 2004 WTPFP, a containment wall around both coagulant tanks has been constructed that can adequately handle the 12,000-gallon volume from the coagulant tanks. The other chemical that is used frequently that could warrant a containment system is the area around the filter aid polymer tanks. The polymer tanks have a combined volume of 580 gallons.

Visual structural deficiencies were noted in the concrete masonry unit (CMU) blocks on the east wall. Due to a gap between the basin wall and the CMU wall, water has started to seep through and cause the CMU blocks to deteriorate. The west wall is comprised of cast-in-place concrete on the bottom portion with CMU blocks starting where basin 3 is located. Water has started leaking through the CMU blocks on the west wall. Deterioration of the plaster architectural material and corroded process piping was observed by the south entrance floor. See the Photo Log, Appendix D – Chemical Storage and Maintenance Room for additional details and photos.

Sedimentation Basins

Basin 1 was built as part of the original plant construction in 1931 and is over 80 years old. Basins 2 and 3 were added to increase plant capacity in 1950 and 1983, respectively. The concrete in basin 3 appears to be structurally sound with few observed cracks in the exterior walls. Basins 1 and 2 have started to show degradation and cracks in their walls. The launders in all basins show little sign of deterioration and are in fair condition. The Structural and Seismic Report details the structural deficiencies and improvements that would need to be made to the older basins as part of the baseline improvements.

Filters

Filters 1, 2, and 3, were built as part of the original plant construction in 1931 and are over 80 years old. Filters 4 and 5 were added in 1950. Filters 6, 7, and 8 were added as part of the most recent plant expansion project in 1983. Structurally, the filters appear to have many years of remaining useful life.

As discussed in Chapter 2 and Chapter 4, many improvements to the existing filter media and underdrains were performed in 2005 that increased plant production efficiency and ensured

continued compliance with water quality regulations. At the time the improvements were made, the consensus industry opinion was that the shallow media filters would not need air scour. However, accumulation of solids on the filter walls that need to be periodically manually cleaned has suggested a need for air scour to further improve filter performance efficiencies. In addition, the task of manually cleaning the filter walls is dangerous for workers and presents an elevated risk of injury. Also affecting filter performance is the location of the filter effluent flow meters which does not allow the measurement of filter-to-waste flows. This condition prevents monitoring of the filter flow during initial startup and hinders the transition from filter-to-waste to filter production.

Filter Galleries

The filter galleries and equipment have been maintained very well and most of the observed deficiencies can be attributed to general wear in line with the WTP's age. Routine maintenance has been performed to address leaks, valve problems, and other equipment maintenance needs and repair cycles. Despite this, the gallery for filters 1, 2, and 3, and the gallery for filters 4 and 5 include piping that lacks proper seismic and structural support. Some piping is in need repair or replacement of insulation or repair of exterior corrosion protection coating, or both. In addition, conduits were placed behind an architectural wall cladding, making modifications difficult. This wall cladding is deteriorating in many areas. See the Photo Log, Appendix D – Filter Galleries for additional details and photos.

Clearwell

The 433,000-gallon clearwell, which serves as a wetwell for the high service and backwash pumps, and a contact basin for disinfection, appears to have significant structural deficiencies. The clearwell is actually comprised of three interconnected areas which were built at different times. These three areas are referred to as the East, Center, and West clearwells and are located under each group of filters and the lobby of the plant. A common filtered water channel currently routes all filtered water to the east clearwell where it is chlorinated. It then flows through a series of serpentine baffles through the center and west areas and finally to the finished water pump area in the west clearwell.

Additional clearwell volume should be added if the plant's capacity is increased, preferably to provide a minimum of 1 hour of detention time at peak flow, but with enough volume to provide for successive filter backwashes at approximately 35,000 to 55,000 gallons each without compromising disinfection performance.

High Service Pump Station

The high service pump station includes high service pumps, backwash pumps, and an air compressor system. Each of these are reviewed in this section.

High Service Pumps

The high service pump station includes two large pumps, two medium pumps, and one small pump, installed in 1961, 1983, and 1983, respectively. The sizes of these pumps are presented in Chapter 4. The high service pump station is currently rated for a firm capacity of 16.7 mgd with a maximum pumping capacity of 21 mgd with all five pumps operating. All of the pumps and motors have been rebuilt within the last 15 years and the City has budgeted for at least one pump and motor rebuild over the next five years. With continued maintenance and repair, the pumps appear to be capable of continued service throughout the planning horizon considered for this report. With future pump upgrades, the existing pump station would be able to supply up to 30 mgd to the distribution system.

Backwash Pumps

The original backwash pump was installed in 1983 as part of the plant expansion project. A back-up backwash line connected to the high service discharge header was also installed, but has never been used due to a lack of pressure and flow control. The backwash pump has required little maintenance according to plant staff, and appears to be functioning appropriately. The pump and system should have significant remaining useful life.

An additional backwash pump was installed in 2012, and the connection to the high service discharge header was removed. This modification allows the 1983 pump to be removed and serviced. During the process, however, it was discovered that the check valve on the pump discharge line is failing and will need to be replaced.

Air Compressor System

The plant is equipped with two air compressor and receiver systems located in the high service pump room. Both systems provide plant air to operate the pneumatic valve actuators for the filters, as well as providing air to keep the surge tank pressurized. Both systems have required little maintenance and appear to be functioning properly. Plant staff is considering relocating the air compressor systems to a different area of the plant to aid in general operations. It is expected that these systems have many years of useful life remaining, although they may not be required in the future if the pneumatic valve actuators are replaced with electric actuators. See the Photo Log, Appendix D – High Service Pump Room for additional details and photos.

Flow Meters and Flash Mix Vault

Both the raw water and finished water pipelines are equipped with Venturi type flow meters. The backwash flow is measured using an electromagnetic type flow meter. The pressure-sensing tubing associated with a Venturi type meter is prone to collecting air bubbles, significantly decreasing the accuracy of the meter. Replacement of the raw and finished water flow meters with electromagnetic type meters is recommended when the budget will allow.

The flash mix vault currently uses static mixing to disperse chemicals. Drawings and specifications of this system appear to be missing from plant records. Since the plant is able to operate at such a wide range of flows, static mixing most likely does not provide the most efficient chemical dosing and mixing. Replacement with a pressure diffusion system, addition of another static mixer, or another approach could positively impact chemical usage. The capital improvement alternatives developed in later chapters discuss mixing options.

Major Valves and Actuators

Most pneumatic actuators at the plant were installed prior to 1980, except those installed in filters 6, 7, and 8 during the most recent plant expansion. All pneumatically operated filter valve actuators are old and in need of repair. Replacement parts for these actuators are becoming increasingly difficult to obtain. Replacement of these actuators with modern electric valve actuators for ease of control and maintenance is recommended. All of the existing air piping in the filter galleries would be removed as part of the actuator replacement project. This would create space in the galleries and allow for better access to the existing equipment.

Several valves, including the filter influent valves and the backwash valves, currently leak and are in need of replacement or repair. Installing new valves with the actuator replacements is recommended since the valves are relatively inexpensive compared to the electric actuators. The City will also benefit from warranties if new valves are provided with new actuators. These improvements should be made in conjunction with other filter gallery piping and flow meter improvements and general maintenance such as removing plaster, relocating electrical conduits mounted on the walls, and painting walls and floors.

Washwater and Solids Handling

The equalization basin contains three transfer pumps which deliver washwater and solids to the old mill pond. The two smaller pumps were installed as part of the 1983 expansion and the larger pump was installed after as part of another project. Largely due to the rough conditions of service that these pumps experience, they are all at the end of their remaining useful lives. When replacing these pumps, the City should consider installing at least one pump with a higher capacity to increase the overall pumping capacity and reliability. However, doing this would increase space demands, as pump configurations provided by manufacturers have changed since the installation of the existing pumps. This may require some challenging new configurations of piping and pump layouts.

Depending on the long-term strategy for solids handling at the plant, improvements to the old mill pond and the washwater equalization basin may be required. Improvement alternatives for solids and washwater handling are presented in Chapter 6. See the Photo Log, Appendix D – Solids Handling for additional details and photos.

Water Quality Testing and Monitoring Facilities

The plant uses on-line water quality instrumentation and bench-top equipment to monitor and control plant performance. Raw water turbidity is continuously monitored using a HACH Surface Scatter on-line analyzer. Settled water turbidity from each basin is also continuously monitored using individual HACH 1720E turbidimeters to assist in process optimization. Each filter is equipped with a HACH 1720E on-line turbidimeter to monitor filter performance and ensure regulatory compliance. A similar on-line turbidimeter is installed on the high service pump station discharge header pipe to continuously monitor the combined filtered water quality exiting the plant. All turbidimeter signals are integrated into the SCADA system. Installation of individual particle counters on the filter effluent is recommended to better predict turbidity breakthrough in the future and ensure continued regulatory compliance.

The plant is equipped with an on-line finished water pH analyzer (HACH pHd) to continuously monitor the plant effluent pH to monitor for corrosion control compliance. Raw water and settled water pH are measured periodically each day via grab samples analyzed in the plant's laboratory.

One HACH CL-17 on-line chlorine residual analyzer is used to monitor the plant effluent residual from the high service pump station discharge header. Pre-basin and settled water chlorine residuals are measured periodically each day via grab samples. The plant's laboratory appears to be equipped with adequate bench-top analytical equipment to perform routine daily testing for monitoring and control, but lacks storage space and would benefit from having additional bench-top space to make working in the lab more efficient. See the Photo Log, Appendix D – Laboratory for additional details and photos.

Plant Drain Sump Pump

The plant has a common sump which collects discharge from the drains of the basins during cleaning operations. The discharge is collected in the sump and pumped to the washwater equalization basin. Currently, cleaning operations are hindered by the sump pump because it is undersized. Plant staff is considering a near-term improvement to replace this sump pump with a larger pump. If a new clarification process is constructed, basin cleaning will be done differently and the size of the sump pump will not be as critical.

Instrumentation and Control Systems

The plant has a Windows-based SCADA and control system that is operated via a central computer station. The existing control systems were installed as part of the SCADA improvements in 2002, and should have some remaining useful life. Recent upgrades at the WTP include new processors, software updates, and upgrading from wires and cables to fiber optic communications. When new systems and equipment are added to the plant, the existing SCADA system will need to be modified and integrated accordingly.

As technology continues to evolve, the SCADA system at the plant will likely require additional software and firmware upgrades. During the planning horizon considered for this report, it is anticipated that replacement software and hardware will be needed to stay current with developing technology. These improvements and upgrades should be made via operating budget investments at the appropriate time and are not included as part of any near-term capital investments included in this Plan.

Electrical Systems

The plant's electrical power is provided via a 1,500 kVA main transformer located on the plant site. The electrical service and transformer were upgraded during the 1983 plant expansion project. The existing plant electrical service and transformer appear adequate to provide service over the next 20 years as demands increase to 20 mgd. Improvements to the electrical system capacity and service need be addressed as part of future expansion projects or if major new electrical loads are added prior to the expansion.

The plant has not experienced any prolonged or severe power outages over the past 20 years. During normal power outages, service has been restored within 1 to 2 hours, with a few cases where power has been out for 16 to 24 hours. This historical level of power service is expected to continue, but there is no guarantee that the City will not face an extended power outage in the future.

Some water treatment facilities are equipped with emergency power sources, such as generators, which can allow a minimum level of water production in the case of an extended power outage by the service provider. Some water providers also have dual electrical feeds from different parts of the power grid to reduce the risk of an extended outage. The City is currently in the process of procuring a back-up power generation project that will provide 5 mgd of water production capability at the plant in a power outage.

In 2010, all interior lighting was upgraded with new ballasts and high-energy-efficiency bulbs. However, this project did not accommodate the outdoor lighting. The current outdoor lighting is not adequate for a plant that operates close to 24 hours per day on a semi-regular basis. The staff often needs to use portable lighting outside to perform tasks. This is inefficient and can pose safety hazards. A project to upgrade outdoor lighting should be included in this plan as a new capital improvement.

Plant staff has responded to increasing security needs by keeping the main gate to the plant normally closed during all hours. Since there is no intercom or other communication system between the gate and the plant, this can cause difficulties for visitors, deliveries, and vendors. Some type of communication system is recommended for the main gate to improve security and operator efficiency.

Control Attic

The existing HVAC system does not provide efficient climate control. Temperatures are often too hot in the summer and too cold in the winter. The existing HVAC control panel has become dated to the point that it is difficult to find technicians to perform work on the equipment and find parts. The system must be operated manually and does not take advantage of the energy savings that could result from a more modern, programmable system. Improvements to update heating and cooling systems in the control and break rooms located within the control attic are recommended.

There is currently limited space available for storage and maintenance or repair within the control attic and throughout the plant. As the plant grows or equipment is replaced, space requirements will increase. Improvements to increase the available storage and working space at the WTP are recommended. See the Photo Log, Appendix D – HVAC for additional details and photos.

Plumbing

Drains at the plant should be inspected to confirm their service and discharge connections to the sewer, the old mill pond, or wastewater equalization basin, and to determine if they are damaged or in need of repair. Most observed plant drains appear to be undersized for their service areas. A recent leak in the high service pump station room caused water to pool into the electrical room and could have potentially caused an electrical failure at the plant.

Other Code Compliance Issues

A cursory review of the WTP was completed to assess conformance to current regulatory codes and standards including seismic and structural integrity, building code conformance, OSHA, and ADA compliance. This information will help identify further needs and planning-level costs associated with future improvements.

A review of the construction documents for the WTP confirmed that the major WTP structures were designed and constructed prior to the consideration of current seismic loads, leaving them susceptible to significant damage or potential collapse in the event of a strong ground motion earthquake. There have been several earthquakes in the Pacific Northwest over the past 20 years that could have severely damaged the WTP had they occurred in proximity to Grants Pass. Anticipated improvements as part of this project include installation of pipeline restraints and reinforcement of concrete structures, especially the older basins and filters.

The walkways around the filters and basins are protected by guardrails. The spacing between horizontal railings may be too large to meet current OSHA requirements. No improvements are recommended at this time.

The plant access and pathways do not meet current ADA requirements. The City should work with local building officials to establish the degree of ADA compliance necessary at the WTP during preliminary design of improvements. Project costs to make the existing plant fully compliant with ADA standards are not included in costs developed with this planning document.

Summary

Table D-2 in Appendix D presents a detailed discussion of major deficiencies identified at the existing WTP. The following list provides a summary.

- Results of structural testing have shown that concrete and rebar in basins 1, 2, and 3 are generally in good condition. The concrete in the East and West clearwells is showing some degradation. Based on these tests and previous inspections and analyses of the existing plant structures, it is believed that the remaining useful life can be in excess of 40 years if seismic and structural upgrades are implemented.
- All chemical feed systems are in relatively good condition and can reliably meet the City's needs for many years. However, this equipment has a finite useful life, and will need to be replaced once within the planning horizon considered for this report. The replacement schedule will depend on when the equipment was installed and is hard to predict, so is shown as a longer-term CIP item within the CIP. The City may also need to replace or upsize chemical feed systems if the plant capacity is expanded.
- In order to improve operator safety and communications, improvements to outdoor lighting and additional perimeter cameras for the main gate are recommended.
- The location of the filter effluent flow meters prevents the measurement of filter-to-waste flows which results in potential operations and water quality problems. The existing flow meters lack adequate lengths of upstream and downstream straight pipe, significantly reducing the accuracy of the meters. Therefore, replacement of the filter effluent flow meters is recommended along with piping changes to integrate filter-to-waste flow measurement.
- As technology evolves, the SCADA system at the plant will likely require additional upgrades. During the planning horizon considered for this report, replacement hardware and software will be needed to stay current with developing technology.
- The City could consider improvements to the HVAC system to provide efficient climate control; temperatures are often too hot in the summer and too cold in the winter. Currently, the server room is not climate-controlled. To protect sensitive computer equipment, addition of ventilation is recommended.
- There is currently limited space available for storage and maintenance on the WTP property. The plant currently rents a storage unit off-site from the facility to supplement its on-site storage. As plant demands increase, storage requirements

for dry chemicals will increase, exacerbating the storage limitations. Improvements to increase the available storage space at the plant site are recommended.

CHAPTER 6
FACILITIES PLANNING CRITERIA

Introduction

This chapter establishes planning criteria and introduces capital improvement alternatives to address the deficiencies summarized in the preceding chapters. The objective of all of the capital improvement alternatives is to enable reliable long-term water supply from the City's Rogue River source, meeting both demand and water quality requirements.

Planning Criteria

Planning criteria for developing capital improvement alternatives include the planning period, water demand projections, a pre-screening of treatment process alternatives, and considerations for redundancy and water supply reliability. Each of these criteria is discussed in the following sections.

Planning Period

Selection of an appropriate planning period for facility upgrades is critical to establishing hydraulic and process capacity requirements. In order to complete an equitable comparison of all possible capital improvement alternatives, the planning period must be the same for each alternative. Factors that affect selection of the planning period include:

- Life expectancy of new or existing civil, mechanical, and electrical equipment needed with the upgrade.
- Life expectancy of any new or existing structure being designed or integrated as part of the upgrade.
- Capacity limitations due to water rights, required space, or other restrictions that are likely to remain fixed for the planning period duration.
- Capacity limitations of existing infrastructure planned to remain.
- Other design considerations, such as desired level of treatment redundancy and nominal capacities of individual treatment trains.

It is anticipated that construction of a new WTP would begin in five to seven years, allowing time for potential property acquisition, design, environmental and regulatory permitting, public acceptance, financing, bidding, construction, and commissioning. Construction of improvements at the existing plant might begin sooner.

For facility replacement cost planning, a life expectancy of 20 to 30 years is often used for equipment with electrical, hydraulic, or mechanical support systems. Facilities such as pipe, concrete basins, and buildings are expected to last longer, with a minimum life expectancy of 75 years. If construction begins in 2020, these facilities would be expected to last until 2095. Therefore, the planning period for all alternatives is through 2095.

Water Demand Projections

To properly size the upgrades for all processes and transmission facilities, water demand projections must be established. The design flow for WTP capacity is normally MDD for water utilities that have adequate distribution system storage. Using MDD as the capacity criteria for upgrades within this Facility Plan Update is consistent with methodologies used in previous Grants Pass planning documents and adheres to State of Oregon and AWWA guidelines.

Table 6-1 summarizes future water demand projections developed as part of the April 2013 MSA technical memorandum titled Long-Term Water Demand Projections which is included as Appendix E. Development of these water demand projections considered existing service area, future service areas, and trending of historical population and water demand information. As recommended in the technical memorandum, these demands should be re-evaluated at regular intervals to account for changing conditions.

Chapter 4 establishes a current WTP capacity of approximately 20 mgd, which is estimated to meet MDD until year 2028. Therefore, the recommended immediate need for capital improvements is based less on capacity expansion and more on the condition and operational constraints of existing facilities.

The City recently made significant improvements to its raw water intake structure allowing for an ultimate intake capacity of 30 mgd. With seismic and structural upgrades, it is anticipated that the structure would be suitable for use through 2065 when system MDD reaches 30 mgd. The cost of upgrading the existing structure is lower than the cost of permitting and constructing a new intake. Constructing other facilities to an initial capacity of 30 mgd allows a consistent criteria for evaluating alternatives and maximizes use of the existing intake structure. Providing 30 mgd capacity is adequate to meet the City's projected MDD through year 2065. For the purposes of this study, the planning capacity for initial construction of all other WTP elements is chosen to match the capacity of the intake for the development of all improvement alternatives. The ultimate design capacity is chosen to be 45 mgd anticipating that new structures and buildings will have design life of 75 years.

Treatment Process Pre-Screening

This section presents a pre-screening of treatment processes considered for WTP improvement alternatives. Design criteria for appropriate treatment technologies used in the alternatives are discussed further in Chapters 7 and 8. Included as part of the pre-screening process is the nature of the Rogue River's source water quality, current and anticipated future water quality regulations, and the City's historical plant operation experience.

Table 6-1
Grants Pass Water Demand Projection Summary

Year	Service Area Population	AAGR[1] (percent)	Per Capita Demand (gpcd[2])	ADD[3] (mgd)	MDD[4] (mgd)
2015	38,632	2.1	170	6.6	15.5
2020	42,862	2.0	170	7.3	17.1
2025	47,323	1.9	170	8.0	18.9
2030	51,993	1.8	170	8.8	20.8
2035	56,844	1.7	170	9.7	22.7
2040	61,843	1.6	165	10.2	24.0
2045	66,951	1.5	160	10.7	25.2
2050	72,125	1.5	155	11.2	26.3
2055	77,700	1.5	150	11.7	27.4
2060	83,704	1.5	145	12.1	28.5
2065	90,173	1.5	140	12.6	29.7
2070	97,142	1.5	140	13.6	32.0
2075	104,650	1.5	140	14.7	34.4
2080	112,738	1.5	140	15.8	37.1
2085	121,451	1.5	140	17.0	40.0
2090	130,837	1.5	140	18.3	43.0
2095	140,948	1.5	140	19.7	46.4
2100	151,841	1.5	140	21.3	50.0
2105	163,576	1.5	140	22.9	53.8
2110	176,218	1.5	140	24.7	58.0

Notes
1. Average annual growth rate
2. Gallons per capita per day
3. Average day demand
4. Maximum day demand

Clarification

Water pumped from the river intake is called raw water and it is pumped from the river to sedimentation basins. Clarification is performed ahead of filtration and usually makes use of chemical coagulation. The clarification process removes a sufficient portion of sediment from the raw water to allow for an efficient filtration process. A conventional WTP uses separate chemical mixing, flocculation, and sedimentation facilities prior to filtration. The three sedimentation basins at the existing Grants Pass WTP currently fulfill this role. However, no flocculation process precedes the sedimentation basins after rapid in-line chemical mixing and none of the basins were designed for optimal hydraulic flow. In addition, chemical injection and mixing equipment is not optimal for the full range of plant flows. Cumulatively, this reduces the effectiveness of the clarification process and requires increased maintenance associated with basin cleaning and residuals removal. It also causes periodic increased solids loading on the filters which results in more frequent backwashing. The City's WTP has three sedimentation basins, each with a unique configuration and size.

Flow splitting between these existing basins is used because of their distinctly different hydraulic and treatment characteristics. Flow splitting is accomplished by visual observation of basin levels and manual valve throttling.

The performance and structural analysis of these basins found seismic deficiencies in all three basins, in addition to high maintenance needs associated with frequent manual basin cleaning. Given the visible cracking and structural degradation of basins 1 and 2, and the short-circuiting that occurs in basin 3, these basins are at the limits of their design life and are in need of replacement.

There are a number of potential clarification process alternatives including:

- Conventional mixing, flocculation, and sedimentation
- Solids contact and sludge blanket clarification
- Dissolved air flotation
- Ballasted flocculation
- No clarification

Table 6-2 shows advantages and disadvantages associated with these clarification processes, all of which would include chemical addition for coagulation. These factors are used to determine whether the technology is appropriate for further analysis. As shown in Table 6-2, both conventional clarification and ballasted flocculation are considered in capital improvement alternatives. These two technologies also present a range of planning-level cost considerations and space requirements associated with clarification.

Ozone

Ozone is a strong oxidant used for disinfection as well as taste and odor control. It can also be used in combination with granular activated carbon filter media to provide biologically active filtration that promotes multiple water quality benefits, including the removal of trace organic compounds. While water produced at the City's WTP has high overall water quality, occasional taste and odor events have occurred in recent years. The potential influence of climate change within the Rogue River watershed could result in more frequent algal blooms and increased taste and odor concerns. The addition of ozone to the treatment process will minimize the occurrence and severity of these events. The preferred location for ozone contact is between clarification and filtration, referred to as intermediate ozonation. Ozone technology is considered in all improvement alternatives as a potential future technology.

Filtration

The eight existing filters at the Grants Pass WTP all have identified seismic issues. Concrete deterioration and cracking of filters 1 through 5 have been observed, though not to the degree of the older exterior sedimentation basins. With an investment in retrofitting work that includes both seismic restraint and concrete basin rehabilitation, additional filter life can be

Table 6-2
Summary of Clarification Process Alternatives

Clarification Process	Advantages	Disadvantages	Screening for Further Consideration
Conventional sedimentation preceded by mixing and flocculation	• Proven treatment technique for Grants Pass • Multiple processes offers level of operational flexibility • Low equipment cost • Higher rate sedimentation can be offered through installation of inclined settlers	• Large footprint required • Higher cost associated with basin construction • Inadequate space at existing WTP site for 30 mgd	Considered at a new WTP site where space is less restrictive
Solids contact and sludge blanket clarification	• Smaller footprint than conventional sedimentation • Lower chemical use	• High operator attention required with changed water conditions • More mechanical components • Higher power costs associated with recirculation • Can take longer periods to achieve effective treatment at start up • Not commonly used	Not considered, other technologies offer higher clarification rates and reduced footprints
Dissolved air flotation	• Smaller footprint than conventional sedimentation • High clarification rate achievable • Lower chemical use	• High operator attention required • More mechanical components associated with skimming • Higher power costs associated with aeration • Not suited for turbid waters which contain silts and settleable solids	Not considered, other technologies offer higher clarification rates and reduced footprints
Ballasted flocculation	• Very high clarification rates achieved • Lowest footprint required • Lower overall capital cost than conventional • Increased recent popularity	• High operator attention required • More mechanical components associated with flocculation and sand addition • Higher power cost than conventional sedimentation and flocculation	Considered for existing WTP upgrades and new WTP construction
No clarification	• Smaller footprint and cost savings • Reduced operator time	• Direct filtration would require large clearwell/ additional disinfection time • Might create disinfection byproduct issues • Rogue River water quality not conducive to direct filtration	Not considered because direct filtration of Rogue River water would create an undue increase in maintenance associated with downstream facilities

achieved, though not the same design life of newly constructed filters. Retrofitting work may not eliminate the limitations on operational efficiencies that treatment processes currently experience as the analysis has found that hydraulics and filter media depth will limit filtration rates and a lack of air scouring can limit backwashing efficiencies.

Capital improvement alternatives include existing filter rehabilitation scenarios and new filter construction scenarios. The rehabilitation alternatives will be based on achieving a life expectancy sufficient to last through 2065 when system MDD reaches 30 mgd. This capacity has been determined as the maximum attainable capacity at the existing WTP site.

Filtration alternatives considered as part of this plan include:

- Mixed granular media
- Deep-bed granular media
- Low-pressure membranes
- Slow sand filtration
- Diatomaceous earth

Table 6-3 lists advantages and disadvantages associated with these filtration processes to identify whether the technology is appropriate for incorporation into capital improvement alternatives. Membrane filtration is a relatively new technology that comes at a premium but consistently produces high quality water. However, membranes do not usually perform well on water from turbid sources such as the Rogue River. As such, membrane technology is not considered to be a good candidate for Grants Pass because the Rogue River source water would require clarification before the membranes. The construction of clarification prior to membranes, which is not typical for membrane installations, makes the cost of this technology prohibitively high. For this reason, granular media filtration, including standard and deep-bed configurations, is the only technology incorporated into capital improvement alternatives.

Solids Dewatering and Residuals Handling

Until ten years ago, no dewatering was performed at the Grants Pass WTP. All solids accumulated through basin cleaning or backwashing cycles were eventually passed along to the old mill pond. Pond dredging and frequent hauling of solids residuals from the pond for disposal became increasingly expensive. Geomembrane bags have since been used effectively to reduce the amount of solids delivered to the pond, but this practice requires a lot of space and labor. The need for solids handling will only increase as demands increase, and this geomembrane method of dewatering could lead to obstacles in meeting NPDES discharge permit requirements for the outfall from the pond. Hauling fees for disposal in the near future might also be subject to increases associated with more stringent permitting.

Table 6-4 summarizes the advantages and disadvantages associated with common solids handling technologies. With reduced footprint either being required or desired, and with the

Table 6-3
Summary of Filtration Process Alternatives

Filtration Process	Advantages	Disadvantages	Screening for Further Consideration
Mixed granular media	• Proven treatment technique for Grants Pass • Lower equipment and capital costs compared to membrane filtration	• Lower filtration rates • Larger footprint required	Considered
Deep bed granular media	• Higher filtration rates available • Smaller footprint required	• Higher capital cost to construct deeper filters • Filter efficiency might decrease slightly compared to shallow media	Considered
Low-pressure membranes	• Consistent high quality water • Physical barrier against waterborne pathogens • Lower chemical use for coagulation • High level of redundancy	• High operator attention associated with control and testing/cleaning support systems • Very high capital cost • Higher operational costs • Cost prohibitive where savings in reduced clarification facilities cannot be achieved	Not considered, due to probable need for a clarification process and pre-screening to protect the membranes (cost prohibitive)
Slow sand filtration	• Simple, reliable technology • Low operator attention required • Low equipment cost	• Very low filtration rates • Requires longer ripening period at startup • Very large footprint required makes it prohibitive for both existing or a new property • Not appropriate for “live” rivers with turbidities > 10 NTU	Not considered due to regulatory constraints and raw water turbidities
Diatomaceous earth	• Lower equipment and chemical costs	• High operator attention associated with pre-coating process and frequent media changes • Not commonly used, very seldom for large capacity facilities • More expensive than granular media filters	Not considered because the media is not readily available

Table 6-4
Summary of Solids Handling Alternatives

Dewatering Process	Advantages	Disadvantages	Screening for Further Consideration
None	• Minimizes number of treatment facilities	• Recent operations indicate that the capacity limitation of the old mill pond renders this alternative as an undesirable high risk, cost-prohibitive alternative	Not considered because of risk and cost
Drying beds	• Simple technology • High percent solids can be achieved with adequate space and weather	• For solids production levels at 30 mgd, space prohibitive at either the existing or a new property	Not considered because space is too limited
Geomembranes	• Portable technology • High percent solids can be achieved with adequate space and weather	• Polymer needed • Space prohibitive at 30 mgd • Labor intensive • Extended process might result in future old mill pond discharge permit compliance issues	Not considered – this current practice is too labor-intensive and will take up too much space at future production levels
Mechanical	• More compact footprint • More automated process, less labor involved • High percent solids achievable without weather conditions • Reduction in hauled volumes	• High initial capital costs • Polymer and power needed • More mechanical equipment	Considered

possibility of increased disposal and permitting costs, mechanical dewatering is the appropriate technology to use with all capital improvements. The process typically uses dewatering equipment preceded by thickening.

As long as the City can continue to discharge liquid residuals from the old mill pond to Skunk Creek, then continued use of the pond to receive spent filter backwash water is considered feasible. Spent backwash water contains relatively low solids concentrations compared to residual streams produced by the clarification process.

Recycling of Residual Streams

The City does not currently recycle any liquid waste streams, and existing WTP site constraints might make it more challenging to use recycling alternatives at that site. No cost or space provisions are included in any of the capital improvement alternatives for liquid residual stream recycling. It may be beneficial to consider recycling larger residual streams at some future date, especially if a new WTP is constructed. Future increases in demand and potential NPDES discharge permit requirements might make continued use of the old mill pond too costly to continue. A brief evaluation of sending residual streams with a large volume to the wastewater collection system found that this is not a viable alternative.

The Filter Backwash Recycling Rule (FBRR) discussed in Chapter 3 regulates recycling of filter backwash, thickener supernatant, and water from dewatering. These streams must be re-introduced upstream of chemical addition for coagulation so that the water undergoes full treatment through the plant. Filter-to-waste is not regulated by the rule because it is typically of high quality and has been filtered, but it is often economical and practical to combine it with the other recycled streams to minimize capital improvement expenditures and operational complexities associated with recycling.

The FBRR does not require treatment of recycle streams as long as they are introduced into the plant ahead of all of the main treatment processes. However, some plants and states have found it beneficial to treat recycled water because it may contain higher levels of pathogens than raw water. The decision to treat recycled streams is usually made on a case-by-case basis between the utility and the regulatory agency.

If recycling were implemented, an equalization storage facility for the various streams would be required to control the recycle flow stream back to the front end of the treatment facility. Often, flow control is best managed by pumping. Equalization basins may be constructed with a common wall, or some other means of redundancy, to facilitate relatively infrequent manual cleaning of settled solids in the basins. Additional space might also be needed if future treatment of recycle streams is necessary.

Using an ultimate design capacity of 45 mgd, typical recovery rates and the potential recovery volumes that might be achieved by recycling the various waste streams under either alternative are shown in Table 6-5. The recovery rates are offered as general industry ranges.

The performance of the City's WTP will vary depending on actual conditions and plant operations.

Table 6-5
New Water Treatment Plant Waste Stream Recycling Summary

Waste Stream	Typical Volume (Percent of Production)	Regulated under FBRR	Potential Recovery at 45 mgd Production (mgd)
Spent Filter Backwash Water	2 to 5	Yes	0.90 to 2.25
Gravity Thickener Supernatant	0.07 to 1	Yes	0.03 to 0.45
Mechanical Dewatering Pressate	0.1 to 0.2	Yes	0.05 to 0.10
Filter-to-Waste	≈ 0.5	No	0.23
Total Potential Recovery from All Waste Streams			**1.21 to 3.03**
Total Potential Recovery from Filters Only (Backwash and Filter-to-Waste)			**1.13 to 2.48**

Chemical Systems

For the purposes of this plan, it is assumed that chemical systems associated with new facility construction will be proportionally similar in configuration and space requirements to existing chemical facilities. Cost estimates and space requirements will be included in improvement alternatives for multiple coagulant (alum, ACH, or PACl) injection systems, a filter aid, thickening agents, and chlorination. Although there are alternative systems associated with each of these chemical processes, such as on-site hypochlorite generation in lieu of 12.5 percent solution delivered, the cost differential between them is not considered consequential in the analysis of alternatives. Space requirements for the largest chemical systems are included in the analyses, as well as additional space for potential future ozonation and pH adjustment equipment systems.

Redundancy Considerations

Designing a WTP to provide redundancy such that the plant could still produce at MDD capacity with any one treatment train for each process off line or out of service comes at a significant capital investment. Additionally, redundant facilities would be underused under normal operating conditions. In practice, most planned facility shutdowns are operationally triggered and can be scheduled during non-peak production periods. Redundancy strategies used in the development of capital improvement alternatives include:

- Backup power supply through an on-site emergency generator sufficient for production of average day winter demands.

- Additional hydraulic capacity in basins and pipelines to provide operation at increased production rates for individual treatment trains over short periods of time.
- No redundancy in raw water pumping facilities to achieve a capacity of 30 mgd. There will be a minimum of six pumps required to meet this capacity, each rated at approximately 5 mgd. Space is available at the intake for no more than six pumps of this size.
- Full redundancy to meet MDD for other pumping facilities that represent critical plant operations including chemical injection, finished water service, and backwash pumping. Full redundancy of filters is also planned, as they represent a critical plant operation.
- No clarification and disinfection basin redundancy to meet MDD. Under this assumption, if only two treatment trains are planned for 30 mgd of clarification, more than 50 percent of MDD could be achieved from either train by running it at higher loading rates for short periods of time. Better raw water quality is anticipated to typically occur during periods of the year which coincide with peak demands. A clearwell which provides adequate CT is compartmentalized, so it is possible to remove portions of the clearwell from service for cleaning and inspection. This work can also be scheduled during a low demand period where CT is still adequately met.

Space Provisions

For new facilities design, several factors beyond the actual required square footage of the treatment process need to be considered in determining adequate treatment facility site footprint including:

- Space for support systems associated with the treatment process, such as air supply, electrical, chemical, HVAC equipment, and mechanical equipment.
- Adequate workspace for operational access to equipment and basins for purposes of inspection and maintenance.
- Code requirements, including building, electrical, mechanical, fire, and plumbing codes.
- Staffing areas such as offices, lunch areas, lockers, restrooms, meeting rooms, and administrative storage.
- Equipment storage and maintenance areas for tools and spare parts associated with treatment plant operations.
- Adequate vehicle access and parking, including consideration of ingress and egress and turning radiuses for large delivery trucks, as well as construction vehicles that will be needed to support future facility maintenance.
- Site designated land uses, setbacks, and consideration of identified critical areas.

Space provisions associated with these considerations are discussed further in the alternatives developed in Chapters 7 and 8, and general site plans are developed.

Capital Improvement Alternatives Overview

Five capital improvement alternatives were developed to represent a full range of potential space, cost, and risk scenarios that address the identified WTP deficiencies and promote reliable, long-term supply of the Rogue River source of supply. The capital improvement alternatives include two existing plant upgrade scenarios and three new plant construction scenarios. The alternatives are as follows:

- Alternative 1: Existing Water Treatment Plant Upgrade, Maximize Reuse of Existing Facilities
- Alternative 2: Existing Water Treatment Plant Upgrade, Phased Replacement of Facilities
- Alternative 3: Construct a New Water Treatment Plant with Consolidated Footprint
- Alternative 4: Construct a New Water Treatment Plant with Large Footprint
- Alternative 5: Construct a New Water Treatment Plant with Consolidated Footprint on Property that the City Already Owns

Chapter 7 discusses the development of Alternatives 1 and 2 which propose improvements at the existing WTP. Chapter 8 discusses the development of Alternatives 3, 4, and 5 which propose construction of a new WTP at a new site.

Other Alternatives

Two other alternatives were initially considered. These alternatives are discussed in this section.

Baseline Alternative

The baseline alternative proposes to make the required structural and seismic upgrades to all of the existing plant structures. A new clearwell and high service pump station would be constructed to enable continued water supply to the distribution system while the existing clearwells and high service pump station are renovated. The cost of these improvements is approximately $12.5 million.

This alternative defers capital investments necessary to expand the plant's capacity and extends the useful life of the existing facilities. The initial capital investment is smaller than that of other alternatives, but the lifecycle cost of this alternative is higher for the following reasons:

- Some of the structures that would initially be renovated would be demolished during later improvements needed to increase plant capacity. A significant portion of the investment to renovate those structures would be wasted.

- The existing plant would still operate inefficiently so annual operations and maintenance costs would continue to be higher than other alternatives.

In addition, this alternative does not address long-term capacity needs or structural longevity needs beyond year 2065, when a capacity of more than 30 mgd is needed. Due to existing property size and constraints, a new WTP with a capacity of 45 mgd would need to be built at a new location. The approximate cost of this new WTP is $75.4 million (2013 dollars). Because of the inherent economic and operational challenges associated with this alternative, it was not evaluated any further.

Peaking Facility Alternative

Another alternative which was initially considered proposes to continue use of the existing plant as a "peaking facility" during peak demand periods. The City would construct a new plant with a capacity of 10 to 15 mgd capable of providing off-peak system demands with provisions to expand capacity up to 45 mgd in the future. The intent of this alternative is to minimize investments in the existing plant and to use the new plant throughout the year as a baseline production facility.

This alternative may have lower initial costs than other alternatives, but it presents major risks and challenges. In addition, the City would need to operate two separate facilities for four to five months every year, requiring additional staff and higher operations and maintenance costs. The existing plant would be "mothballed" every fall and re-started every spring which also presents additional costs and challenges.

Based on preliminary discussions with City staff, it was decided that this option was not due any further analysis. The main reason for this decision was to avoid the need to hire additional plant staff and to avoid the additional annual costs which would be incurred. The higher life cycle cost from the additional labor costs was deemed to be high enough to exclude this option from further consideration.

Summary

A summary of the WTP improvement planning criteria established in this chapter is shown in Table 6-6. The planning criteria summarized in this chapter serve as a basis for development of the capital improvement alternatives discussed in Chapters 7 and 8.

Table 6-6
WTP Improvement Alternatives Planning Criteria Summary

Item	General Criteria Adopted for Improvement Alternatives
Capacity of Structures	30 mgd initial, 45 mgd ultimate
Capacity of Equipment	30 mgd or less initially, deferment as appropriate to save life expectancy, 45 mgd ultimate
Design Life Expectancy of New Equipment	20 to 30 years minimum
Design Life Expectancy of New Structures	75 years minimum
Design Life Expectancy of Refurbished Structures	45 years
Clarification Processes Considered	Conventional clarification, ballasted flocculation
Filtration Processes Considered	Granular media, standard or deep-bed, high-rate
Solids Handling Processes Considered	Mechanical thickening and dewatering
Chemical Systems	Largest alternative space requirement for each, provisional space for ozone and pH adjustment
Full Redundancy	Hydraulic capacity, finished water service and backwash supply pumping, chemical injection pumping, filtration
Partial Redundancy	Emergency power supply, raw water pumping, clarification, disinfection

CHAPTER 7
EXISTING WATER TREATMENT PLANT ALTERNATIVES

Introduction

This chapter presents alternatives which can expand capacity and address the existing deficiencies at the existing WTP site. Planning level project cost estimates are also presented which are used for comparison with other alternatives.

The ultimate capacity at the existing WTP site is limited to 30 mgd. This is due to space limitations at the site and raw water intake capacity. Therefore, in order to meet long-term water demands, a new treatment facility with a new intake would still be required in approximately 2065 according to Chapter 6 findings. The capacity of the new facility could vary depending on the degree of investment initially made in new facilities at the existing site. Two alternatives for expanding the existing plant were developed for this facility plan; the alternatives were intended to bracket the spectrum of options with regard to cost.

Alternative 1 Overview

Alternative 1 retains as many existing process facilities at the existing WTP site as practicable at a lower granular media filtration rate. For this alternative, the existing filter building, clearwells, and high service pumping facilities are retained and upgraded. A new clarification process facility uses ballasted flocculation to reduce the treatment process footprint and a mechanical dewatering process is added. Three additional filters and new clearwell systems are constructed to provide 30 mgd capacity. Alternative 1's approach attempts to use as much of the older existing structure as possible. A significant capital investment would be required for this alternative once 30 mgd is exceeded in year 2065. A new 45-mgd treatment facility would then need to be constructed and the existing 30 mgd plant would be abandoned. This alternative involves a higher level of risk associated with extending the life of existing deteriorated facilities, some of which have reached the end of their useful design life.

Alternative 2 Overview

Alternative 2 replaces most of the facilities at the existing WTP using phased demolition and reconstruction. This alternative adds new ballasted flocculation and dewatering facilities and demolishes the existing filter building. As part of this alternative, deep bed, high rate filters would be constructed. A new clearwell and high service pump station would also be constructed. Because all critical facilities under this alternative are newly constructed, the life expectancy of the resulting 30 mgd WTP would be approximately 75 years, through the end of the planning period evaluation. In 2065, when system MDD reaches 30 mgd, a new WTP and intake will be needed to supplement the existing WTP and achieve a total capacity of 45 mgd between the two plants. The City would operate two plants through the end of the planning period resulting in higher operating costs than operating a single WTP.

Site and Construction Constraints

Implementing either alternative requires consideration of the risks and challenges related to construction activities around the existing plant and within the limited available space. Since this is the City's only water supply, construction activities at the existing plant site must not interfere with on-going operations and the production of safe drinking water. These alternatives are subject to the following additional planning criteria:

- To implement the required improvements, the maximum plant production rate will likely be reduced for extended periods of time, as basins or filters are taken out of service for repairs or demolition. Based on recent historical water system demands which show a peak week demand and peak month demand of approximately 11.5 and 10.5 mgd, respectively, a maximum plant production rate of 10 to 12 mgd may be tolerable as construction activities occur. The City may need to implement water use restrictions or rationing during hot weather conditions to limit demands based on plant production limitations with facilities out of service.
- The WTP cannot be shut down for more than two to three consecutive days during the low-demand period from November to April. Shutdowns which last for one day may be tolerable during October and May. No plant shutdowns are acceptable from June to September.
- The most pressing short-term concern is structural and seismic rehabilitation of the east and west clearwells. The 1980s clearwell, which also serves as a wet well for the high service pumps, also requires structural and seismic upgrades. All of these clearwell upgrades will take longer than three consecutive days to complete. Therefore, other plant additions must be completed, such as another clearwell and a new high service pump station (HSPS), before upgrades can be completed on the 1980s clearwell.
- Existing high-voltage power lines run north-to-south over the western part of the site, limiting construction activities which can be performed directly beneath. Locating permanent facilities directly beneath the power lines would present significant construction and permitting challenges.
- There is a 50-foot set-back between the edge of the property and above-grade structures.
- Construction activities cannot hinder vehicular traffic around the plant perimeter, including chemical delivery traffic.
- The space required for construction staging and storage exceeds the available space at the existing plant site. This limitation could result in added construction costs for both alternatives.
- A phased construction program may be considered to spread out capital investments, but extended construction duration will have an impact on plant operations and project costs.

- While there is no immediate need to expand production capacity in order to meet near term water demands, there is an immediate need for seismic and structural upgrades.

Process Alternatives and Selection

This section presents the basis for developing the two alternatives introduced in this chapter. Each of the primary processes and main support facilities are briefly discussed below.

Intake and Raw Water Pump Station

The existing intake was retrofitted with a fish screen system in 2008 and has a hydraulic capacity of 30 mgd. The four existing raw water pumps are a vertical turbine configuration which withdraws water from a wetwell downstream of the screens. The raw water pump station (RWPS) has space provisions for two additional pumps to expand from 20 mgd to 30 mgd. The intake and adjacent riverbank requires structural upgrades and stabilization to protect against failure resulting from a seismic event. Beyond 2065, an adjacent second intake structure and associated pumping and transmission facilities will be required to bring intake capacity to 45 mgd. The proposed improvements to the intake and RWPS are similar for both alternatives.

Rapid Mixing

A new rapid mixing system that provides for optimum chemical dispersion will be required to expand the plant capacity to 30 mgd. The new system will need to be located in the new raw water pipeline to be constructed between the existing RWPS and the new ballasted flocculation system. There are a number of rapid mixing systems considered as part of this assessment, including:

- In-line static mixer, similar to the existing system
- In-line mechanical mixer
- Pumped diffusion system

For planning purposes, a new pumped diffusion system is recommended since it provides optimum mixing over a wide range of flows, reduces chemical use and head loss, and uses less energy than the other options.

Clarification

The combined area occupied by basins 1, 2, and 3 is approximately 21,000 square feet. Basins 1 and 2 have an approximate combined hydraulic capacity of 12 mgd and basin 3 has an approximate hydraulic capacity of 8 mgd. As discussed in Chapter 2, basin 3 does not perform as well as basins 1 and 2 due to its square configuration and radial flow design.

Basin 1, 2, and 3 Improvements

Without significant structural and seismic improvements, basins 1 and 2 are approaching the end of their useful lives. Even with these improvements, their hydraulic capacity cannot be increased enough to provide an additional 10 mgd of clarification capacity. As part of earlier studies completed in 2009 concerning installation of a settled solids collection system, it was estimated that the project cost of the recommended improvements for basins 1 and 2 would be approximately $3 million and would not result in additional capacity. Additional costs are required for basin 3 to make structural and seismic upgrades and to improve its performance. In order to increase its clarification capacity to 30 mgd using similar technology, an additional large basin would be necessary. In total, all of these improvements would likely cost in excess of $7 million to achieve a capacity of 30 mgd and to extend the useful life of the older basins by approximately 45 years.

New High-Rate Clarification

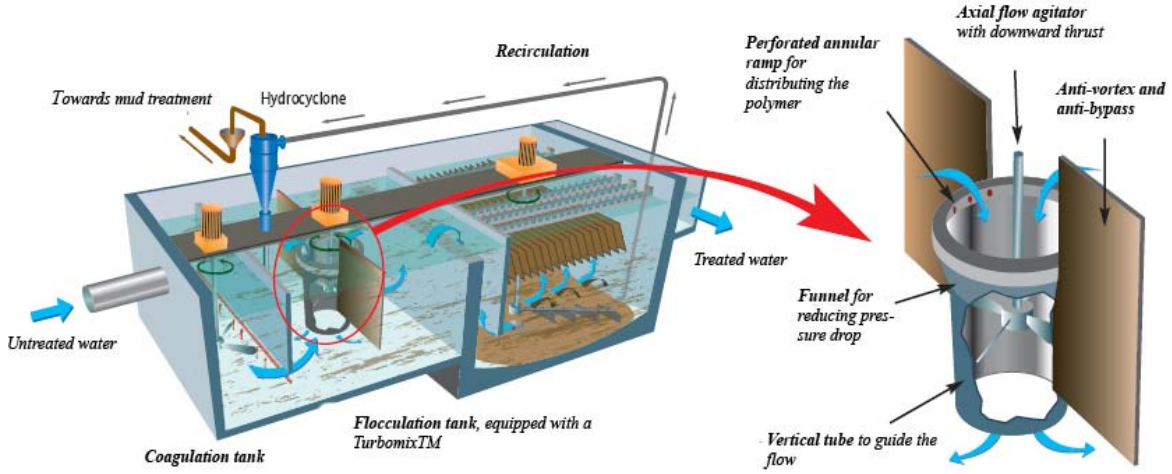
An alternative to improving the existing clarification structures is the construction of a higher-rate clarification system which will reduce surface area requirements. As discussed in Chapter 6, potential high-rate processes include:

- Tube settlers or plate settlers preceded by mechanical flocculation
- Upflow sludge blanket clarifiers
- Dissolved air flotation
- Ballasted flocculation

Based on the consultant team's experience elsewhere in the region, a new high-rate ballasted flocculation process could be constructed to replace the three existing basins and provide an optimized clarification system. This technology has gained considerable acceptance in Oregon and throughout the country over the past decade, and it is believed to be appropriate for treatment of the Rogue River raw water. The process uses settling rates of 20 to 30 gpm/ft^2, compared to settling rates of 1 to 6 gpm/ft^2 for other clarification processes, and can be constructed on a very small footprint. Figure 7-1 presents a schematic overview of this process.

The required footprint for ballasted flocculation at 30 mgd capacity is approximately 2,100 square feet which is smaller than any one of the existing basins. Basin 1 or 3 could be taken off-line for a season and demolished. Then, ballasted flocculation could be constructed in that space. The two remaining basins could then be re-purposed or demolished to create space at the site for other improvements. Ballasted flocculation and its associated costs are not expected to be any higher than the costs of improving the existing basins and building and constructing an additional basin to get to 30 mgd capacity. The new ballasted flocculation structures will have a 75-year expected useful design life.

Figure 7-1
Schematic of the Actiflo™ Turbo Ballasted Flocculation Process



(Courtesy of Kruger, Inc.)

Ballasted flocculation to provide 30 mgd capacity would consist of two 15-mgd process trains. This would provide operational flexibility and redundancy and would be less expensive than installing three process trains with individual capacities of 10 mgd. It would also require that a solids thickener be constructed to handle the recycle flows and the solids produced, as well as a sand feed system and a polymer feed system. Improvements to the settled water conveyance system would be required to properly distribute flows to the filters.

Based on recent manufacturer quotes and construction costs elsewhere, it will cost less than $5 million to install a 30-mgd ballasted flocculation system, including thickening, at the existing WTP site. For all of these reasons, ballasted flocculation is recommended as the clarification process for both Alternatives 1 and 2 to expand and upgrade the existing WTP.

Ozone

Space for future ozone equipment is reserved under Alternatives 1 and 2 in case the City decides to implement ozone for taste and odor control, or for any other unforeseen circumstance in the future. Multiple ozone contact basins sized to provide adequate contact time at full capacity could be installed between the clarification and filtration processes with liquid oxygen storage and ozone generators located nearby. Approximately 2 feet of hydraulic head would be needed if ozonation facilities were added in the future, which results in a clarification water surface level that is higher than the current level.

Filtration

As discussed in Chapter 6, it is recommended to continue use of granular media filtration for the Rogue River supply. Low-pressure membrane filtration, the other common alternative, is

more expensive to construct and operate than granular filters. Therefore, both upgrade and expansion alternatives for the existing Grants Pass WTP include granular media filtration.

The eight existing filters at the WTP were upgraded in 2007 and are deemed to have useful life for the next 40 years with a production capacity of 20 mgd with one filter out of service. Some structural and seismic upgrades are required, particularly in the oldest three filters. The filter design is not ideal to meet current filtration standards due in part to a relatively shallow media depth. It is believed, however, that these filters can continue to operate efficiently in the future in conjunction with an optimized clarification system such as ballasted flocculation.

Alternative 1 Filtration

Alternative 1 proposes the continued use of the eight existing filters. To achieve 30 mgd of filtration capacity, the construction of three new filters with a media area similar in size to filters 4 and 5 is recommended. The depth of media can be slightly deeper than the existing filter media to enhance performance. The new filters would be backwashed using the existing backwash pumps inside the HSPS. The location of the new filters should be determined based on available space and proximity to the clearwells and HSPS.

Alternative 2 Filtration

For Alternative 2, it is proposed to demolish all of the existing filters, including the associated buildings and clearwells, and replace them with six new high-rate, deep-bed granular media filters. A maximum filtration rate of 8 gpm/ft^2 with one filter out of service using 48 to 60 inches of dual media is recommended, based on successful experiences elsewhere with similar raw water and clarification systems. This higher filtration rate will reduce footprint and reduce construction costs. The new filters would initially be built and put in service before the existing filters are demolished, so they need to be located in an area that is currently open or available after demolition of the existing sedimentation basins. The new deep-bed filters would be backwashed using new pumps inside the proposed new HSPS and would also be cleaned using an air scour system. It is also possible to consider the use of granular activated carbon as a filter media in lieu of anthracite as an added taste and odor control feature.

Disinfection and Finished Water Storage

The WTP existing finished water storage structures include three separate clearwells built at different times. They have a combined total volume of approximately 433,000 gallons with an operating volume of 362,000 to 400,000 gallons, which is just enough to meet current disinfection requirements using free chlorine under most conditions. Ultraviolet irradiation (UV) disinfection is an alternative to free chlorination for future primary disinfection at the Grants Pass WTP. Implementing UV allows for a reduction in required finished water storage volume required for disinfection, but does result in increased power cost. Because a large finished water storage basin is needed for proper operation of the HSPS, it is believed

that free chlorine will continue to be the most cost-effective alternative for future primary and secondary disinfection. It is also anticipated that disinfection byproduct concentrations will be reduced after improvements are complete, due mainly to the planned discontinuation of pre-chlorination. Future disinfection with free chlorine will need to be achieved downstream of filtration.

If free chlorine remains the primary disinfectant at the Grants Pass WTP, additional clearwell volume will be required. Based on seasonal demands and temperature profiles, at least 650,000 gallons of baffled storage will be required to meet the 0.5-log *Giardia* inactivation requirements at 30 mgd.

As documented in the June 2012, "Structural and Seismic Evaluation Report" presented in Appendix D, the existing clearwell is in immediate need of structural upgrades to minimize the risk of damage during a seismic event. Unfortunately, the estimated duration of implementing these improvements exceeds the maximum allowable plant shut-down duration of three days. Though a temporary UV disinfection system could potentially allow for upgrades to the 1930s and 1960s portions of the clearwell, additional clearwell and distribution pumping capacity will be needed prior to upgrades to the 1980s portion of the clearwell because this is where the existing HSPS is located. Therefore, for Alternative 1, construction of a new 375,000-gallon clearwell and a new HSPS is recommended. The recommended volume is adequate to meet current CT requirements with free chlorine during the clearwell upgrade construction period when operating at flows under 15 mgd. Once the new clearwell and HSPS are put into service, the existing clearwell can be taken out of service and repaired, either all at once or sequentially. Temporary and permanent yard piping improvements will also be required to connect the old and new clearwells.

For Alternative 2, a new 650,000-gallon clearwell is recommended for construction in conjunction with a new 30-mgd HSPS prior to abandoning and demolishing the existing three clearwells.

For both Alternatives 1 and 2, it is recommended that the new clearwell be located directly beneath the new filters and HSPS to minimize footprint and piping.

High Service Pumping

The existing HSPS has approximately 30 mgd of available capacity, assuming that the distribution system is upgraded to receive this additional flow. However, the clearwell below the HSPS requires structural and seismic upgrades, requiring construction of a new clearwell and a 10 to 12 mgd HSPS for Alternative 1 as previously discussed. The discharge piping of this new, smaller HSPS would be connected to the existing plant finished water pipeline. Following the structural and seismic upgrades, this new HSPS would operate in parallel with the existing HSPS, increasing operational flexibility and overall plant reliability.

For Alternative 2, a new 30-mgd HSPS should be constructed along with a new 650,000-gallon clearwell. The proposed location for the new HSPS and clearwell is in the area currently occupied by basins 1 and 2. Hence, these older basins would need to be demolished following construction and startup of the proposed new ballasted flocculation system so that the new HSPS and clearwell can be constructed. When these new facilities have been completed, the existing HSPS and associated buildings can be demolished. The new HSPS would discharge into a new finished water pipeline that would connect to the existing pipeline. The new HSPS would also be equipped with backwash pumps to support the new filters proposed under Alternative 2.

Chemical Storage and Injection

The existing chemical storage area and sodium hypochlorite room has enough capacity to treat more than 20 mgd. It is not anticipated that additional space on the site would need to be dedicated to treat 30 mgd, even when considering the additional polymer system required for the proposed ballasted flocculation system. Chemical metering pumps could be added or replaced to meet the increased chemical feed rate.

Alternative 1 Chemical Storage and Injection

Rather than increasing the number or size of the existing chemical storage tanks, the Alternative 1 approach would be to schedule more frequent chemical deliveries. As such, Alternative 1 will use the existing chemical storage and feed areas, following structural and seismic upgrades, to achieve a remaining useful life of 45 years. The existing maintenance area for the WTP is currently co-located in the chemical storage area. This maintenance area will be moved to a new dedicated space elsewhere on the site for Alternative 1. Space is also available to add a carbon dioxide tank and feed system for raw water coagulation and pH control if it becomes necessary in the future.

Alternative 2 Chemical Storage and Injection

For Alternative 2, the existing chemical storage areas will be replaced with a new chemical building. This new building would be built and put into service before demolition of the existing chemical building. The proposed location for the new chemical building is adjacent to the new filters.

Residuals and Solids Handling

The backwash water from the existing filters is currently equalized in a basin located on the west end of the plant, then pumped to the old mill pond across the street. The suspended solids concentration in this water is relatively low and the pond acts as a settling basin to ensure that the overflow from the pond, which discharges to Skunk Creek, meets NPDES requirements. The pond also provides time for the chlorine residual to dissipate. Because the pond is partially filled with solids deposited prior to the development of the geobag

dewatering program, the City employs a dredging program in the pond every summer to remove and dewater settled solids.

The solids which collect in the three contact basins are removed two to three times per year on a batch basis since the basins do not have continuous solids removal systems. These solids are equalized in an on-site tank and then treated with polymer before being pumped into geobags. The geobags are located to the west of basin 3 and to the north of the equalization basin.

Mechanical Dewatering

Due to limited available space, a mechanical dewatering system is recommended which can remove solids on a continuous basis and produce higher dewatered solids concentrations than the current method. A mechanical dewatering system includes three key components: thickening, storage and equalization, and mechanical dewatering equipment. Dewatering equipment options include belt presses, centrifuges, and screw presses.

The mechanical dewatering equipment and ancillary features should be installed in a building, preferably with the dewatering equipment on the second story to facilitate truck loading for off-site disposal. For both Alternatives 1 and 2, the mechanical dewatering building is located to the west of the main plant facilities and is capable of accommodating any of the mechanical dewatering variations.

Liquid Residuals

As long as disposal of liquid decant from the old mill pond to Skunk Creek remains acceptable, it is recommended to continue handling backwash water using the current practice. Decant from the new gravity thickener would also be pumped to the old mill pond. If Skunk Creek becomes unavailable in the future, then improvements to clarify and recycle the backwash water and thickener decant to the raw water stream upstream of flash mix would likely be necessary. For Alternatives 1 and 2, the existing equalization basin is retained. The pumps would be replaced due to age, and conveyance capacity to the pond would be increased.

Operations and Maintenance Support Facilities

The existing plant provides insufficient space for efficiency and effective operations and maintenance activities. Under Alternative 1, the existing administrative building and facilities will remain and a new maintenance/shop/storage building is recommended for construction. It should be located in available space created by demolition of the existing buildings. In Alternative 2, the existing administrative building will be demolished and a new operations and maintenance building will be built where the existing administrative building and filters 1 to 5 are currently located. Based on preliminary discussions with the City's planning department, the new operations building must be similar in appearance to the existing buildings to match the historical nature of the older plant buildings.

Summary of Alternatives

Both Alternative 1 and 2 site plans for expanding and upgrading the existing WTP include the following primary treatment processes:

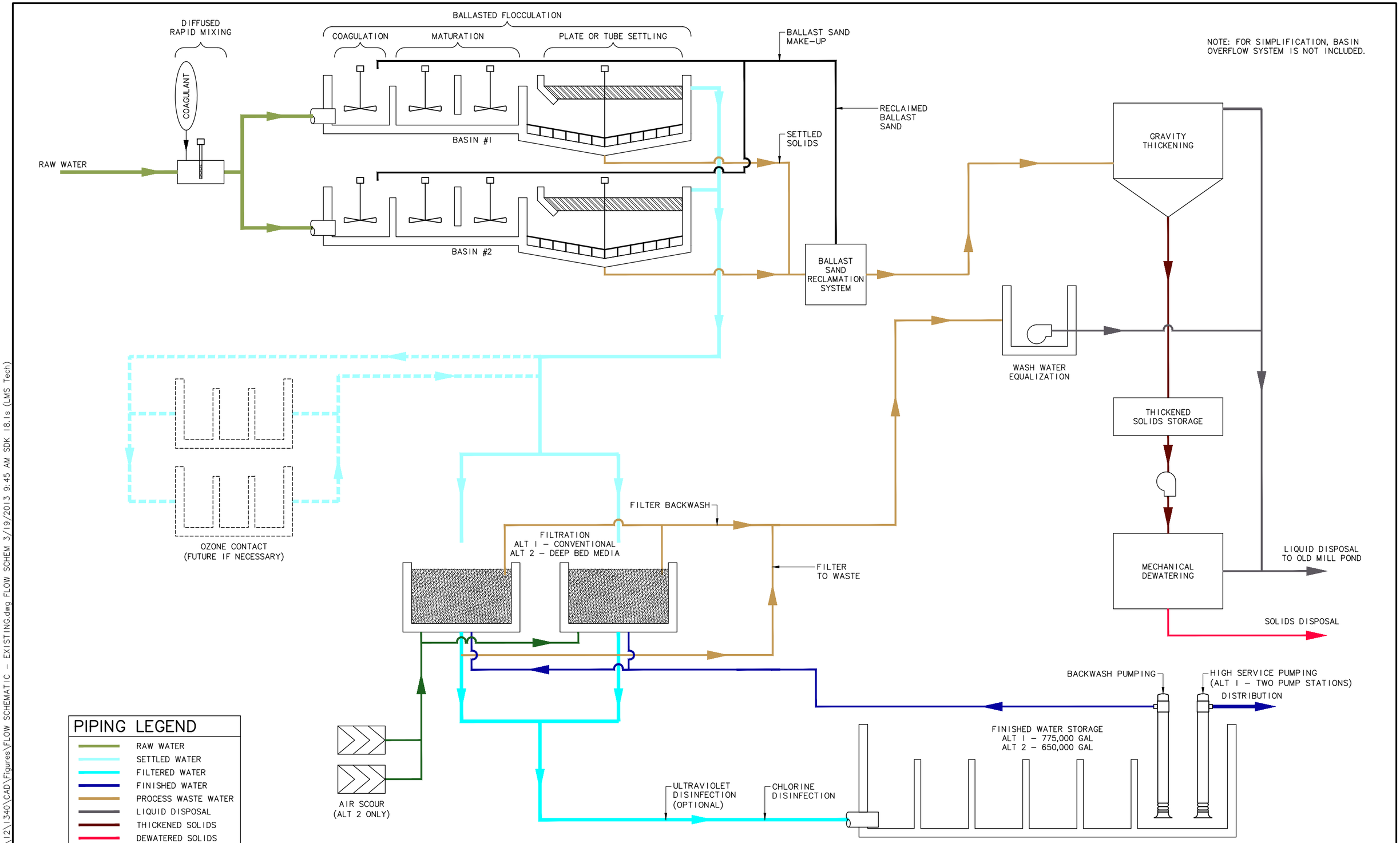
- Pumped diffusion rapid mixing
- Ballasted flocculation
- Potential future intermediate ozonation
- Granular media filtration
- Chlorine disinfection

The solids treatment train includes gravity thickening, solids homogenization, and mechanical dewatering facilities. New high service pumping facilities are also required for both alternatives. In both alternatives, the layout attempts to minimize building footprints and costs where possible. Both alternatives would result in a 30-mgd WTP with a remaining useful life of approximately 45 years. Alternative 2 has a longer useful life as it includes construction of multiple new facilities. Figure 7-2 shows process flow schematics for Alternatives 1 and 2.

Table 7-1 presents a comparison of the advantages and disadvantages for the two alternatives. Either alternative will require the construction of new intake facilities and a new WTP at a different site in approximately 2065 when the water system MDD reaches 30 mgd. The capacity of the future new WTP would be 45 mgd for Alternative 1 and 15 mgd for Alternative 2.

Table 7-1
Existing Water Treatment Plant Expansion and Upgrade Alternative Summary

Alternative	Advantages	Disadvantages
1	• Lowest initial capital cost • Increased HSPS reliability and operational flexibility • Preserves architectural look of historical buildings	• Increased risk of water rationing during construction • Multiple smaller filters with shallow media depth • Requires construction of a new 45-mgd plant in 2065
2	• New facilities offer useful life of more than 45 years • More efficient equipment and support systems • Deeper filter media helps with taste and odor control • Newer facilities provide opportunity to comply with current OSHA and ESA codes	• Longest construction duration and water rationing • Highest initial capital cost • Requires construction of a new 15-mgd WTP in 2065 • Results in operation of two plants beyond 2065



REV	DATE	BY	DESCRIPTION

SCALE: NO SCALE

DESIGNED A. NISHIHARA
DRAWN S. KIRK
CHECKED P. KREFT

MSA

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City of Grants Pass

WATER TREATMENT PLANT FACILITY PLAN UPDATE
FIGURE 7-2
PROCESS FLOW SCHEMATIC - ALTERNATIVES 1 AND 2

Facility Layouts and Construction Sequencing

Figures 7-3 and 7-4 illustrate the proposed facilities layouts for Alternatives 1 and 2, respectively. The size, placement, and timing of facilities shown in the figures reflect the discussion of treatment processes in this chapter.

A phased approach to construction is required for both alternatives in order to sequentially complete key elements of the work, assure adequate plant production capacity, and to spread out the costs. In order for plant staff to become familiar with the operation of new facilities associated with progressive phases of work, each construction phase will be followed by a break from construction.

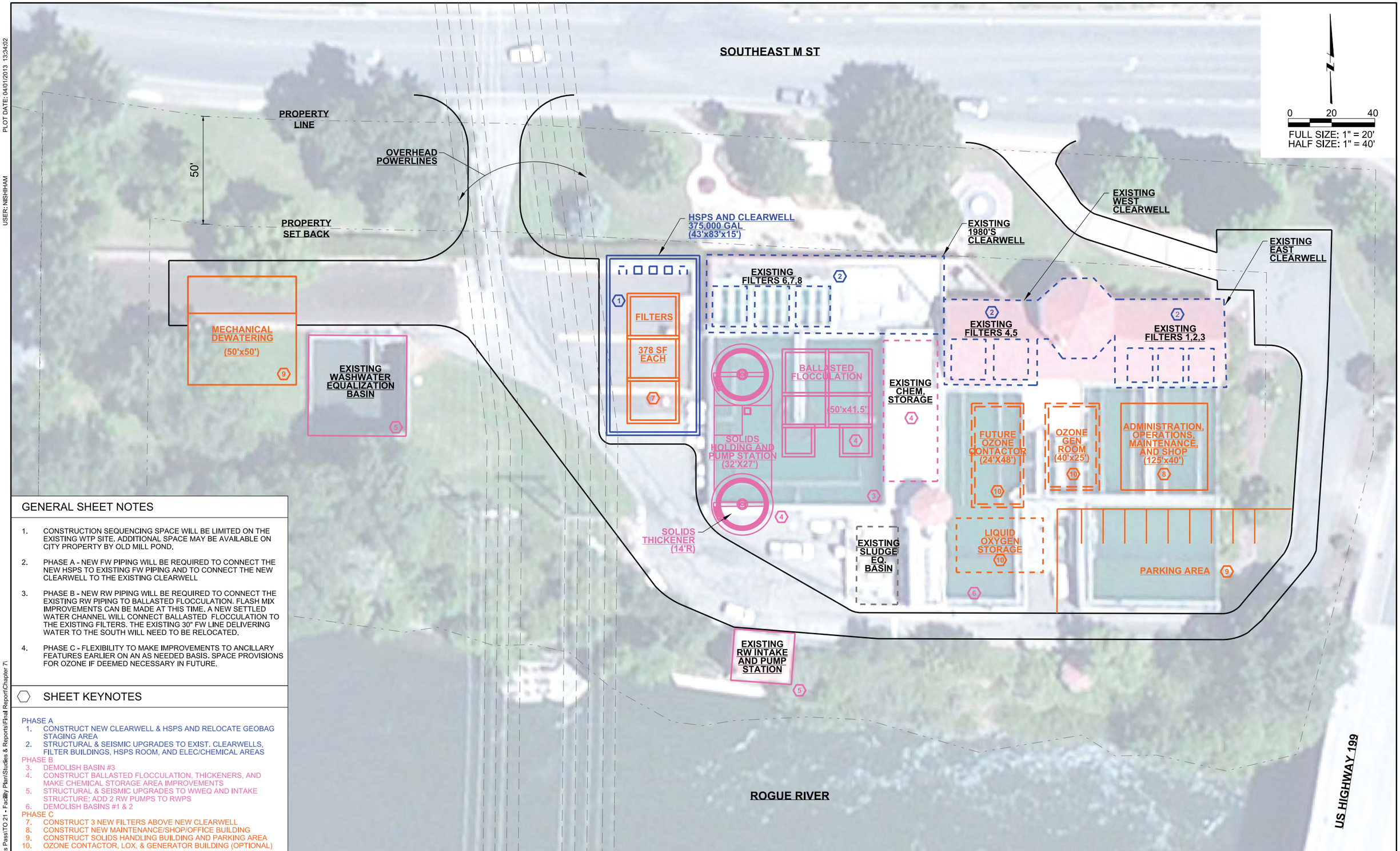
For each alternative, three suggested phases of work with separate construction contracts are summarized in the figure legends. The itemized scope of work for each phase included in the legends is intended to balance the risk of prolonged operational obstructions with addressing the most critical upgrades as early as possible. This staged approach is necessary to maintain plant production and will result in a longer construction duration compared to completing the work under a single construction contract, which will increase costs. Both alternatives will require that the plant operate at a maximum production of 10 to 12 mgd for a period of 12 to 18 months, including at least one summer with potential water rationing, until key facilities can be constructed and brought on-line. For purposes of estimating comparative contractor overhead and profit between alternatives, Alternative 1 is estimated to have three phases. The first phase would have a duration of 12 months and the second and third phases would each last18 months. Alternative 2 is estimated to have three equal phases with a duration of 18 month each.

Construction Constraints

Both alternatives require careful planning and design to implement the proposed improvements and a pre-qualified, experienced contractor with a proven ability to work within the significant site constraints. Discussion of work sequencing, permissible work areas and work hours, and coordination with City operational staff for activities impacting normal plant operations will all need to be included in the construction contract and bidding documents. Construction storage and staging areas need to be established and additional off-site staging may be required. Access to and around the plant site for chemical deliveries and plant activities must be maintained throughout the course of construction activities.

Project Cost Estimates

Table 7-2 and Table 7-3 present planning-level project cost estimates for Alternative 1 and Alternative 2, respectively. The estimated project costs are expressed in 2013 dollars. Due to the phased nature of both alternatives, it is anticipated that these project costs would be incurred in several expenditures over the course of several years. As such, net present value is a more meaningful way to compare the costs associated with these alternatives and the additional alternatives presented in Chapter 8.



GENERAL SHEET NOTES

1. CONSTRUCTION SEQUENCING SPACE WILL BE LIMITED ON THE EXISTING WTP SITE. ADDITIONAL SPACE MAY BE AVAILABLE ON CITY PROPERTY BY OLD MILL POND.
2. PHASE A - NEW FW PIPING WILL BE REQUIRED TO CONNECT THE NEW HSPS TO EXISTING FW PIPING AND TO CONNECT THE NEW CLEARWELL TO THE EXISTING CLEARWELL
3. PHASE B - NEW RW PIPING WILL BE REQUIRED TO CONNECT THE EXISTING RW PIPING TO BALLASTED FLOCCULATION. FLASH MIX IMPROVEMENTS CAN BE MADE AT THIS TIME. A NEW SETTLED WATER CHANNEL WILL CONNECT BALLASTED FLOCCULATION TO THE EXISTING FILTERS. THE EXISTING 30" FW LINE DELIVERING WATER TO THE SOUTH WILL NEED TO BE RELOCATED.
4. PHASE C - FLEXIBILITY TO MAKE IMPROVEMENTS TO ANCILLARY FEATURES EARLIER ON AN AS NEEDED BASIS. SPACE PROVISIONS FOR OZONE IF DEEMED NECESSARY IN FUTURE.

SHEET KEYNOTES

PHASE A
1. CONSTRUCT NEW CLEARWELL & HSPS AND RELOCATE GEOBAG STAGING AREA
2. STRUCTURAL & SEISMIC UPGRADES TO EXIST. CLEARWELLS, FILTER BUILDINGS, HSPS ROOM, AND ELEC/CHEMICAL AREAS

PHASE B
3. DEMOLISH BASIN #3
4. CONSTRUCT BALLASTED FLOCCULATION, THICKENERS, AND MAKE CHEMICAL STORAGE AREA IMPROVEMENTS
5. STRUCTURAL & SEISMIC UPGRADES TO WWEQ AND INTAKE STRUCTURE; ADD 2 RW PUMPS TO RWPS
6. DEMOLISH BASINS #1 & 2

PHASE C
7. CONSTRUCT 3 NEW FILTERS ABOVE NEW CLEARWELL
8. CONSTRUCT NEW MAINTENANCE/SHOP/OFFICE BUILDING
9. CONSTRUCT SOLIDS HANDLING BUILDING AND PARKING AREA
10. OZONE CONTACTOR, LOX, & GENERATOR BUILDING (OPTIONAL)

REV	DATE	BY	DESCRIPTION

SCALE: AS NOTED

DESIGNED A. NISHIHARA
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CHECKED P. KREFT

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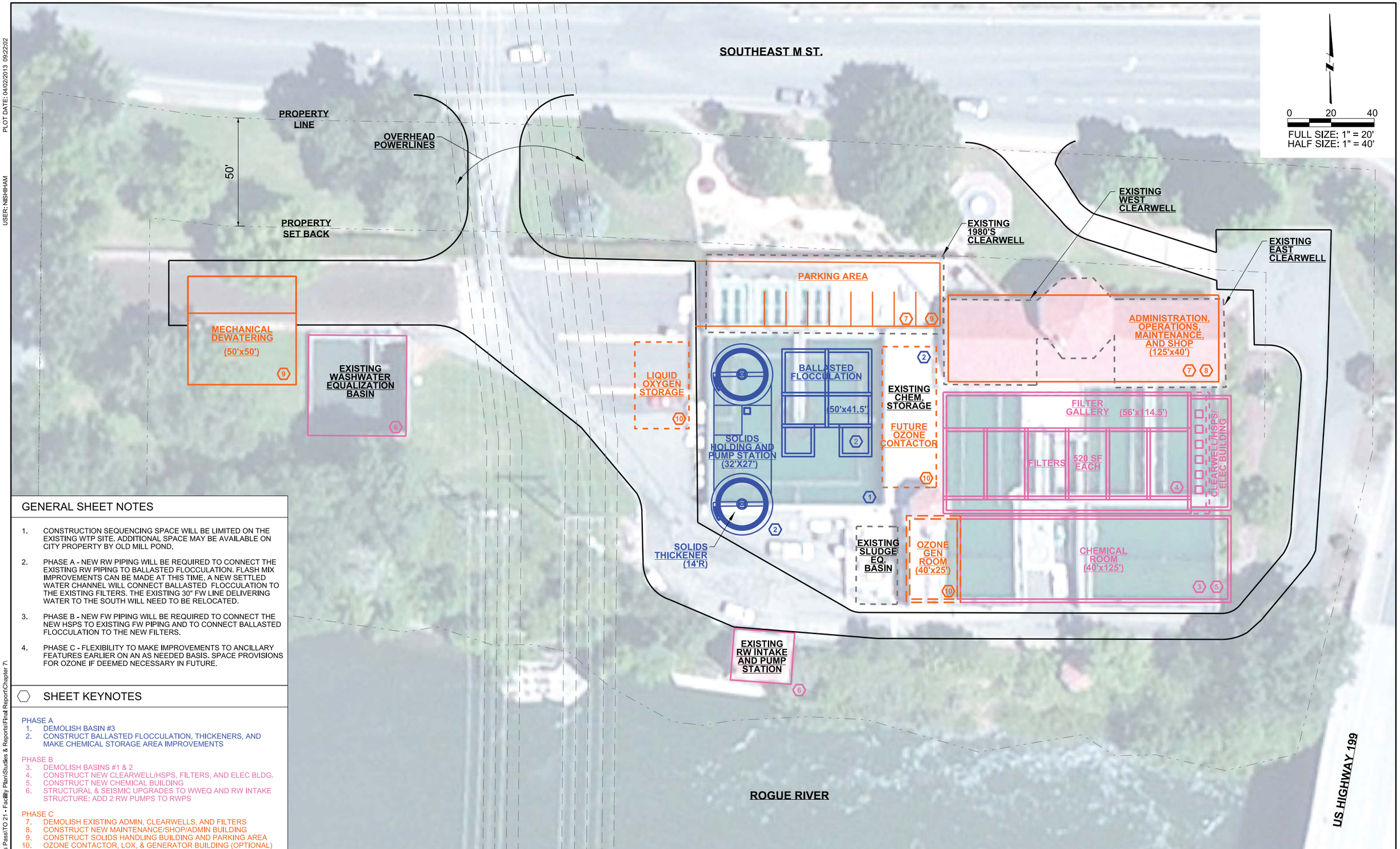
WATER TREATMENT PLANT FACILITY PLAN UPDATE
FIGURE 7-3
SITE LAYOUT - ALTERNATIVE 1



GENERAL SHEET NOTES

1. CONSTRUCTION SEQUENCING SPACE WILL BE LIMITED ON THE EXISTING WTP SITE. ADDITIONAL SPACE MAY BE AVAILABLE ON CITY PROPERTY BY OLD MILL POND.
2. PHASE A - NEW RW PIPING WILL BE REQUIRED TO CONNECT THE EXISTING RW PIPING TO BALLASTED FLOCCULATION. FLASH MIX IMPROVEMENTS CAN BE MADE AT THIS TIME. A NEW SETTLED WATER CHANNEL WILL CONNECT BALLASTED FLOCCULATION TO THE EXISTING FILTERS. THE EXISTING 30" FW LINE DELIVERING WATER TO THE SOUTH WILL NEED TO BE RELOCATED.
3. PHASE B - NEW FW PIPING WILL BE REQUIRED TO CONNECT THE NEW HSPS TO EXISTING FW PIPING AND TO CONNECT BALLASTED FLOCCULATION TO THE NEW FILTERS.
4. PHASE C - FLEXIBILITY TO MAKE IMPROVEMENTS TO ANCILLARY FEATURES EARLIER ON AN AS NEEDED BASIS. SPACE PROVISIONS FOR OZONE IF DEEMED NECESSARY IN FUTURE.

SHEET KEYNOTES

PHASE A
1. DEMOLISH BASIN #3
2. CONSTRUCT BALLASTED FLOCCULATION, THICKENERS, AND MAKE CHEMICAL STORAGE AREA IMPROVEMENTS

PHASE B
3. DEMOLISH BASINS #1 & 2
4. CONSTRUCT NEW CLEARWELL/HSPS, FILTERS, AND ELEC BLDG.
5. CONSTRUCT NEW CHEMICAL BUILDING
6. STRUCTURAL & SEISMIC UPGRADES TO WWEQ AND RW INTAKE STRUCTURE; ADD 2 RW PUMPS TO RWPS

PHASE C
7. DEMOLISH EXISTING ADMIN, CLEARWELLS, AND FILTERS
8. CONSTRUCT NEW MAINTENANCE/SHOP/ADMIN BUILDING
9. CONSTRUCT SOLIDS HANDLING BUILDING AND PARKING AREA
10. OZONE CONTACTOR, LOX, & GENERATOR BUILDING (OPTIONAL)

REV	DATE	BY	DESCRIPTION

SCALE: AS NOTED

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CHECKED P. KREFT

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WATER TREATMENT PLANT FACILITY PLAN UPDATE
FIGURE 7-4
SITE LAYOUT - ALTERNATIVE 2

Table 7-2
Alternative 1 Project Cost Estimate

Facility	Estimated Cost (2013 USD)
Mobilization and General Conditions (12 percent)	$2,500,000
Intake and Raw Water Pump Station Improvements	$1,450,000
375,000 gallons of New Treated Water Storage	$1,000,000
New 7.5 MGD Capacity High Service Pumping Equipment	$1,000,000
New Finished Water Piping	$250,000
Relocate Geobag Staging Area	$100,000
Structural/Seismic Upgrades to Existing 3 Clearwells	$750,000
Structural/Seismic Upgrades to HSPS Room	$100,000
Structural/Seismic Upgrades to Existing Filter Buildings	$500,000
Structural/Seismic Upgrades to Chemical and Electrical Rooms	$400,000
Tank, Electrical Equipment, and Pipe Anchorages	$250,000
Demolish Existing Basin 3	$300,000
Relocate 30-inch diameter Finished Water Pipe	$200,000
New Ballasted Flocculation and Sedimentation	$3,200,000
New Gravity Thickeners and Associated Piping	$900,000
Thickened Solids Storage	$450,000
New Chemical Systems for Ballasted Flocculation and Thickeners	$150,000
36-inch diameter Raw Water Pipe to Ballasted Floc Basins and Flow Splitter	$150,000
Add New Settled Water Channel	$200,000
Demolish Basins 1 and 2	$300,000
Influent Flow Metering and Flash Mix Facilities	$300,000
Build Three New Filters, Retain Filters 1 through 8	$2,600,000
Upgrades to Existing Wastewater Equalization Basin, Pumps, and Piping	$300,000
Mechanical Dewatering Building	$1,350,000
New Maintenance/Shop/Office Building	$2,000,000
Electrical and Instrumentation	$1,500,000
Site Civil and Miscellaneous Yard Piping	$500,000
Landscaping	$50,000
Subtotal: Construction without Contingency	**$22,800,000**
Contingency (25 percent)	$5,700,000
Additional Contractor Overhead and Profit for Three Phases	$2,052,000
Subtotal: Construction with Contingency	**$30,600,000**
Engineering, Permitting, Construction Management Services, Legal, Administration (30 percent)	**$6,840,000**
Total Estimated Project Cost with Contingencies	**$37,400,000**

Table 7-3
Alternative 2 Project Cost Estimate

Facility	Estimated Cost (2013 USD)
Mobilization and General Conditions (12 percent)	$3,700,000
Intake and Raw Water Pump Station Improvements	$1,450,000
Demolish Existing Basin 3	$300,000
Relocate 30-inch diameter Finished Water Pipe	$200,000
New Ballasted Flocculation and Sedimentation	$3,200,000
New Gravity Thickeners and Associated Piping	$900,000
Thickened Solids Storage Tank	$450,000
New Chemical Systems for Ballasted Flocculation and Thickening	$150,000
36-inch diameter Raw Water Pipe to Ballasted Floc Basins and Flow Splitter	$150,000
New Settled Water Channel	$200,000
Influent Flow Metering and Flash Mix Facilities	$300,000
Demolish Existing Basins 1 and 2	$300,000
650,000 gallons of New Treated Water Storage	$1,500,000
New 30 MGD High Service Pump Station	$3,750,000
New Finished Water Piping	$500,000
Build Six New Filters	$5,200,000
Demolish Filters 6, 7, 8, Existing HSPS, and 1980s Clearwell	$500,000
Demolish Filters 1, 2, 3, and East Clearwell	$500,000
Upgrades to Existing Wastewater Equalization Basin, Pumps, and Piping	$300,000
New Administration and Maintenance Building	$2,500,000
Demolish Filters 4, 5, Existing Ops Building, and West Clearwell	$500,000
New Chemical Building	$2,500,000
Mechanical Dewatering Building	$1,750,000
Electrical and Instrumentation	$2,000,000
Site Civil and Miscellaneous Yard Piping	$1,000,000
Landscaping	$100,000
Subtotal: Construction without Contingency	**$33,900,000**
Contingency (25 percent)	$8,500,000
Additional Contractor Overhead and Profit for Three Phases	$3,850,000
Subtotal: Construction with Contingency	**$46,300,000**
Engineering, Permitting, Construction Management Services, Legal, Administration (30 percent)	**$10,200,000**
Total Estimated Project Cost with Contingencies	**$56,500,000**

Project cost estimates were developed using recent local industry information from estimates, bid tabs, vendor quotations, and other material unit costs for similar treatment facilities. Line item estimates represent installed costs that include materials, labor, equipment, and contractor overhead and profit. Building costs for Alternative 2 are higher to account for more expensive architectural finishes that match the look of the existing buildings.

The project cost estimates are Class 5 estimates as defined by the American Association of Cost Engineering. These opinions of probable cost are based on planning-level analysis and a low level of project definition. Accuracy typically ranges from –30 percent to +50 percent.

In developing the project costs for Alternatives 1 and 2, it was necessary to add premiums associated with the risk and difficulty associated with construction at the existing site. Assumptions that were used to develop these costs, which are different from those used to develop costs for the construction of a new WTP, include the following:

- A mobilization cost at a higher percentage than used for construction of a new WTP to account for the potential need by the contractor to secure additional off-site staging areas, and the likely necessity for more heavy equipment transport and storage to minimize on-site contractor presence and impact on ongoing plant operations.
- A higher planning-level construction cost contingency allowance than used for construction of a new WTP, recognizing the increased potential for changed conditions and contractor claims on a confined site with various existing utilities and working constraints.
- Additional contractor overhead and profit assessed when compared to construction of a new WTP intended to account for the cost of on-site equipment and labor proportional to the increased total construction time of all three phases. This is relative to an estimated 30-month construction duration for a new WTP. Contractor overhead and profit was taken as 15 percent of construction cost in this analysis.
- A higher markup used between construction and project costs than used for the new WTP alternatives, accounting for the increased engineering, permitting, construction management, legal, and administrative cost allowances required to administering three separate construction contracts instead of a single contract.

Summary

The two alternatives for expanding and upgrading the existing WTP on the existing WTP site have a wide range of capital costs and have different implications for long-term operation of the City's water supply system over the next century. New WTP Alternatives 3, 4 and 5, presented in Chapter 8, evaluate the construction of new treatment facilities on new sites. A comparative evaluation of all five alternatives, which includes social and environmental considerations as well as costs, is presented in Chapter 9. Chapter 9 includes a final recommendation for the preferred capital improvement program.

CHAPTER 8
NEW WATER TREATMENT PLANT ALTERNATIVES

Introduction

This chapter presents a detailed discussion of alternatives which propose to construct a new WTP at a new site. The decision to investigate replacement alternatives at a new site was made because the cost to retrofit the existing plant is high and the ultimate capacity of any WTP on the existing property is practically limited to 30 mgd. Construction of a new WTP also offers a lower risk profile and more straightforward capacity expansion opportunities when compared to upgrades at the existing WTP. Alternatives 3 and 4 are intended to bracket the spectrum of options with regard to cost and space requirements associated with a new WTP on a new site of unspecified nature. Alternative 5 was developed to investigate construction of a new WTP on a site which is already owned by the City.

Alternative 3 Overview

Alternative 3 proposes construction of a new WTP using newer treatment technologies which have smaller footprints than their conventional counterparts. These processes tend to be more mechanically driven and may require additional regulatory approval. They typically have higher initial equipment costs than traditional treatment technologies, but lower overall costs resulting from smaller basins and structures. The consolidated footprints are used to define the minimum adequate property size that would be needed for a WTP with an ultimate capacity of 45 mgd. It is assumed that initial construction would be for a WTP capacity of 30 mgd, with expansion in 2065 to 45 mgd.

Alternative 4 Overview

Alternative 4 uses conventional treatment technologies which rely on hydraulic residence time for effectiveness. These technologies are proven and accepted by regulatory agencies, but they have a higher capital cost than more recent treatment technologies because they require larger basins and structures. Traditional processes offer some operational flexibility and a degree of reliability that more modern technologies may lack. Larger process footprints associated with conventional clarification and filtration facilities that have lower average flow rates are used to determine minimum property size requirements. Mechanical dewatering is still included by necessity for this alternative, and planned facilities are designed to accommodate an ultimate capacity of 45 mgd. It is assumed that initial construction would be for a WTP capacity of 30 mgd, with expansion in 2065 to 45 mgd.

Alternative 5 Overview

Alternative 5 proposes construction of a new WTP on a property which is currently owned by the City. The property is located across the street from the current WTP property and is currently the site of both the City's skate park and the WTP residuals handling pond. Initial

layouts were completed using both conventional processes and more technologically advanced processes, but it was determined that the property cannot practically accommodate conventional treatment processes and achieve 45 mgd ultimate capacity. The development of this alternative assumes the use of new treatment technologies with consolidated footprints. Since the old mill pond would be filled in under this alternative, additional washwater clarification basins are necessary to handle process wastewater before discharge to Skunk Creek. It is assumed that initial construction would be for a WTP capacity of 30 mgd, with expansion in 2065 to 45 mgd.

Alternative 3, 4, and 5 Planning Principles

The development of new WTP construction alternatives considers some general principles for planning which are different from those associated with the development of Alternatives 1 and 2 as presented in Chapter 7. These considerations include:

- Operations at the existing WTP would continue for the duration of the new plant construction. Production up to the rated 20 mgd capacity of the existing plant would continue to be available during peak periods without the potential need for water rationing.
- The duration of construction for a new WTP is shorter than the duration of construction of improvements at the existing WTP under either Alternative 1 or 2.
- Temporary facilities might be necessary during construction to allow for raw water supply and treated water disposal during startup and commissioning of the new WTP. This may present some disruption to production at the existing WTP, but impacts could be minimized by properly timing the interruptions.
- Construction of a new WTP would not begin as soon as construction of improvements under Alternative 1 or 2 because of the added time required for property acquisition, funding, and potentially more extensive permitting requirements.
- Site layout and construction sequencing of a new WTP are not subject to the constraints of Alternatives 1 and 2.
- Site access, internal traffic flow, parking, visual appeal, and the final site layout would be better optimized with a new WTP.

Property Considerations

For the purposes of connecting a new WTP to the existing water distribution system infrastructure, it is best to locate a new WTP in close proximity to the existing plant. The large-diameter distribution system piping in the vicinity of the existing plant can be used to adequately convey plant flows without significant upgrades. In addition, the existing raw water intake could be reused without major modification, and the old mill pond could continue to be used for process water discharge unless needed for other facility siting, as in Alternative 5. The cost and time to integrate a new treatment plant increases significantly with more distant sites because of pipeline construction costs and potential electrical

infrastructure upgrades. Other challenges associated with a distant site include right-of-way acquisition, environmental permitting for a new intake, liquid waste stream handling, and additional engineering for needed pipelines and electrical infrastructure.

The scope of this study does not include the identification of a specific site for a new WTP under Alternatives 3 and 4. A cursory review of City GIS property, land use, topography, and critical areas information suggests that there are several viable properties within ½ mile of the existing plant. Without knowing specific property characteristics, the most useful methodology for developing new plant alternatives is to cover a full range of potential space and cost requirements at the conceptual level which meet project objectives. Alternative 3 represents the smallest reasonable footprint and Alternative 4 represents the largest reasonable footprint. The treatment process selections bracket cost ranges subject to the planning criteria presented in Chapter 6.

The property used for Alternative 5 is the parcel across the street from the City's existing WTP. The City already owns this property. In this alternative, the old mill pond would be drained and filled to accommodate construction of new WTP structures on the site. The site is too small to accommodate conventional treatment processes at 45 mgd capacity. Available information regarding the geotechnical conditions at the site suggest that construction of WTP structures on the site will be challenging and more costly than typical construction. The City would also be required to demolish the existing skate park located on the property and rebuild the skate park at another location.

Process Alternatives and Selection

This section presents the basis for developing Alternatives 3, 4, and 5. Each of the primary treatment processes and main support facilities are discussed below.

Intake, Raw Water Pump Station, and Rapid Mixing

Alternatives 3, 4, and 5 propose the same improvements for the intake, raw water pumping, and rapid mixing facilities. As with Alternatives 1 and 2, two additional pumps will be added to the existing intake facilities to expand its capacity to 30 mgd. Upgrades to securely tie the structure back into the riverbank to prevent failure during a seismic event or slide will also be made to the existing intake. As with existing plant scenarios, a new intake would be required for production rates in excess of 30 mgd. A new pumped diffusion system for chemical coagulant addition will be constructed at the new WTP site. Construction of a new WTP at any location requires additional raw water transmission piping to supply water to the new location.

Clarification

Without the space restrictions imposed by the existing site, the City may choose to use clarification technologies other than ballasted flocculation. Two locally proven technologies were selected through Chapter 6 pre-screening; these were conventional flocculation and

sedimentation, and ballasted flocculation. Both processes use flocculation and sedimentation, but the ballasted process uses mechanical mixing, microsand addition, and inclined plate settlers to achieve floc maturation and settling with significantly less hydraulic retention time and surface area. These processes represent the high and low end of acceptable clarification rates per unit of surface area and, consequentially, the lowest and highest required surface areas and resulting footprints.

Alternative 3 Clarification

Ballasted flocculation is proposed in Alternative 3, with a proposed configuration of the equipment identical to the existing plant upgrade alternatives. The ballasted flocculation system and unit size would result in a settling rate of approximately 22 gpm/ft^2 at design capacity.

Alternative 4 Clarification

Alternative 4 proposes conventional flocculation and sedimentation which would make use of long, rectangular basins. The train consists of a tapered flocculation process followed by sedimentation. For sizing purposes, two rectangular basins would initially be constructed to achieve a combined capacity of 30 mgd. Each train would have three flocculation chambers with a detention time of 20 to 30 minutes, and a sedimentation basin sized to have a design surface overflow rate of 1 gpm/ft^2. These criteria represent conservative industry standards and the largest process footprint. Higher overflow rates might be achieved in practice, and could certainly be increased with the addition of inclined settlers to the basins. Evaluating these options could be made part of value engineering work completed during final design.

Alternative 5 Clarification

Ballasted flocculation is proposed in Alternative 5 with the same configuration and design flow rate as in Alternative 3. Conventional flocculation and sedimentation requires too much space for this alternative.

Ozone

As with Alternatives 1 and 2, space provisions are allocated for the new WTP alternatives to allow for the future addition of intermediate ozonation. Multiple contact basins sized to provide adequate contact time at full capacity would be installed between the clarification and filtration processes with liquid oxygen storage and ozone generators located nearby. The hydraulic profile of the new WTP should also allow water surface level differentials between the sedimentation basins and filters to allow for head loss associated with the ozonation process.

Filtration

Granular media filters are incorporated into all new WTP alternatives, as recommended in the pre-screening discussions in Chapter 6. New filter design would allow for air scouring during the backwashing process which is currently unavailable with the existing WTP filters. Air scour will reduce spent filter backwash water volumes and increase cycle durations. Filters would also be initially constructed with a deeper bed of granular media that allows higher filtration rates. A common channel for all clarified water can be used to distribute flow to all filters. With this approach, the number of filters does not need to be equally divisible by the number of clarification treatment trains. For Alternatives 3, 4, and 5, the filter layout is based on sizing each filter area to maintain uniform flow and air distribution while providing an appropriate filtration rate.

Alternatives 3 and 5 Filtration

Alternatives 3 and 5 use ten filters with an area of 440 ft^2 each to meet the 45 mgd capacity at a standard deep-bed filtration rate of 8 gpm/ft^2 with one filter off line. Six filters would be initially constructed to achieve 30 mgd at the same filtration rate without redundancy. This configuration would allow the plant to operate long enough to determine if a higher filtration rate can be used while still adequately meeting performance requirements. Other plants in the region commonly achieve 10 gpm/ft^2 with deep-bed media and optimized clarification upstream of the filters.

Alternative 4 Filtration

Alternative 4 uses a more conservative filtration rate of 5 gpm/ft^2 associated with standard granular media depths. This would require a larger ultimate configuration using twelve filters with an area of 520 ft^2 each, with eight initially constructed. Final design of this alternative might include initial construction of basins that could accommodate a future deep-bed media depth, thereby reducing the number of additional filters needed for an expanded plant capacity of 45 mgd.

Disinfection and Finished Water Storage

Alternatives 3, 4, and 5 assume the use of free chlorine to achieve the most stringent 0.5-log *Giardia* inactivation requirements for post-filtration disinfection. The contact time necessary in the clearwell to meet this disinfection requirement is conservatively based on current chlorination practices, historic seasonal demand and temperature profiles, and a well-baffled clearwell design. The clearwell should also have multiple cells, allowing a cell to be isolated and taken off line for inspection during lower demand periods. This configuration would also allow the clearwell to be operated at lower volumes during lower capacity production periods if a water quality benefit is achieved.

For all alternatives, sizing of the clearwell is based on initial construction of the volume required at the ultimate WTP capacity of 45 mgd. This approach eliminates the risks associated with expansion of this critical facility at a later date.

Different clearwell sizes were used for each alternative. The minimum volume necessary to meet disinfection requirements at 45 mgd is 1.1 million gallons. This size of clearwell is used for Alternatives 3 and 5. The clearwell proposed in Alternative 4 has a volume of 2.0 million gallons, reflecting the more conservative footprint of the new WTP planned in this alternative. Clearwell volume requirements will be determined during preliminary design for the selected alternative.

For all alternatives, the new clearwell is located directly beneath the new HSPS to minimize footprint and piping. Minimal space requirements to allow for the installation of future in-line UV units as an alternative future disinfection approach is also provided with the facility layouts.

High Service Pumping

As described above, Alternatives 3, 4, and 5 propose construction of a new HSPS in an enclosed building above the clearwell. The HSPS building footprint is sized to allow adequate spacing for pipe and support equipment between vertical turbine pump units, which would ultimately provide a firm pumping capacity of 45 mgd. The HSPS will also house the backwash pumps.

Chemicals

Chemical storage space needs and cost estimates are based on similar, comparable treatment facilities using similar treatment processes. Optimal chemical storage tank volumes and configurations would be developed as part of the final design process based on delivery schedules and operational preferences. All three alternatives include space provisions and layouts for chemical systems adequate to meet needs for a capacity of 45 mgd. Chemical systems might include multiple coagulants and filter aid systems, sodium hypochlorite, and future potential pH adjustment and ozonation.

Residuals and Solids Handling

Alternatives 1 and 2 included mechanical dewatering processes and equipment for processing filter backwash, filter-to-waste, and other residuals streams. Alternatives 3, 4, and 5 include similar thickening, storage and equalization, and dewatering facilities. The sizing of these facilities at the new WTP differ in that they are sized for an ultimate production capacity of 45 mgd rather than the 30 mgd capacity used in Alternatives 1 and 2.

Alternatives 3, 4, and 5 include an initial 50-foot diameter gravity thickener with an estimated loading rate of 10 lbs per day per square foot of surface area. Space to construct a second thickener of the same size is included for the future expansion. A storage and

equalization tank for thickened solids, initially sized to handle four days of volume at 45 mgd capacity, is also included to offer operational flexibility. The mechanical dewatering building is sized based on the initial installation of two dewatering units and with provisions for one future dewatering unit, along with conveyance systems and a truck loading bay.

It is assumed that dewatered solids are conveyed by trucks for off-site disposal. An equalization basin is also included on site layouts for liquid process stream storage prior to discharge. For Alternatives 3 and 4, the old mill pond will be retained for clarification and discharge to Skunk Creek. Alternative 5 includes a washwater clarification basin to replace the old mill pond because the space occupied by the pond is required for other facilities.

Support Facilities

New WTP support facilities do not include specialty historic architectural finishing like those required at the existing WTP site. The support buildings under new WTP alternatives are based on layout and configuration of treatment facilities that have similar capacities, staffing levels, and support systems to that of Grants Pass. Project cost estimates for the support buildings are based on estimates developed for these similar facilities and assume CMU block walls and metal roof construction materials. Support buildings and areas are the same for Alternatives 3, 4, and 5 and are presented in Table 8-1. The operations and administration building would include staff work areas such as offices, meeting rooms, lockers and restrooms, lunch room, and records storage.

Table 8-1
Planning-Level Support Building Size Summary

Building	Dimensions (Length × Width × Height) (ft)	Area (ft^2)
Chemical Storage	105 × 60 × 20	6,300
Ozone Generator Room	25 × 75 × 20	1,875
Maintenance and Shop	40 × 60 × 15	2,400
Operations and Administration (Two stories)	60 × 50 × 30	6,000
Electrical Building	40 × 40 × 15	1,600

Summary of Alternatives

The treatment processes and facilities included in the new WTP alternatives offer a planning-level analysis of space requirements and allow a fair value comparison between all of the alternatives. Treatment processes common in footprint and cost between Alternatives 3, 4, and 5 include:

- Rapid mixing
- Finished water pumping

- Solids handling facilities
- Support buildings
- Backup power
- Future space provisions

Treatment facilities that differ in footprint and costs between Alternatives 3 and 5 and Alternative 4 include:

- Clarification
- Filtration
- Finished water storage
- Site civil, including site preparation, paving, yard piping, landscaping, security, etc.
- Distribution system integration, based on feasible sites for the different total property requirements

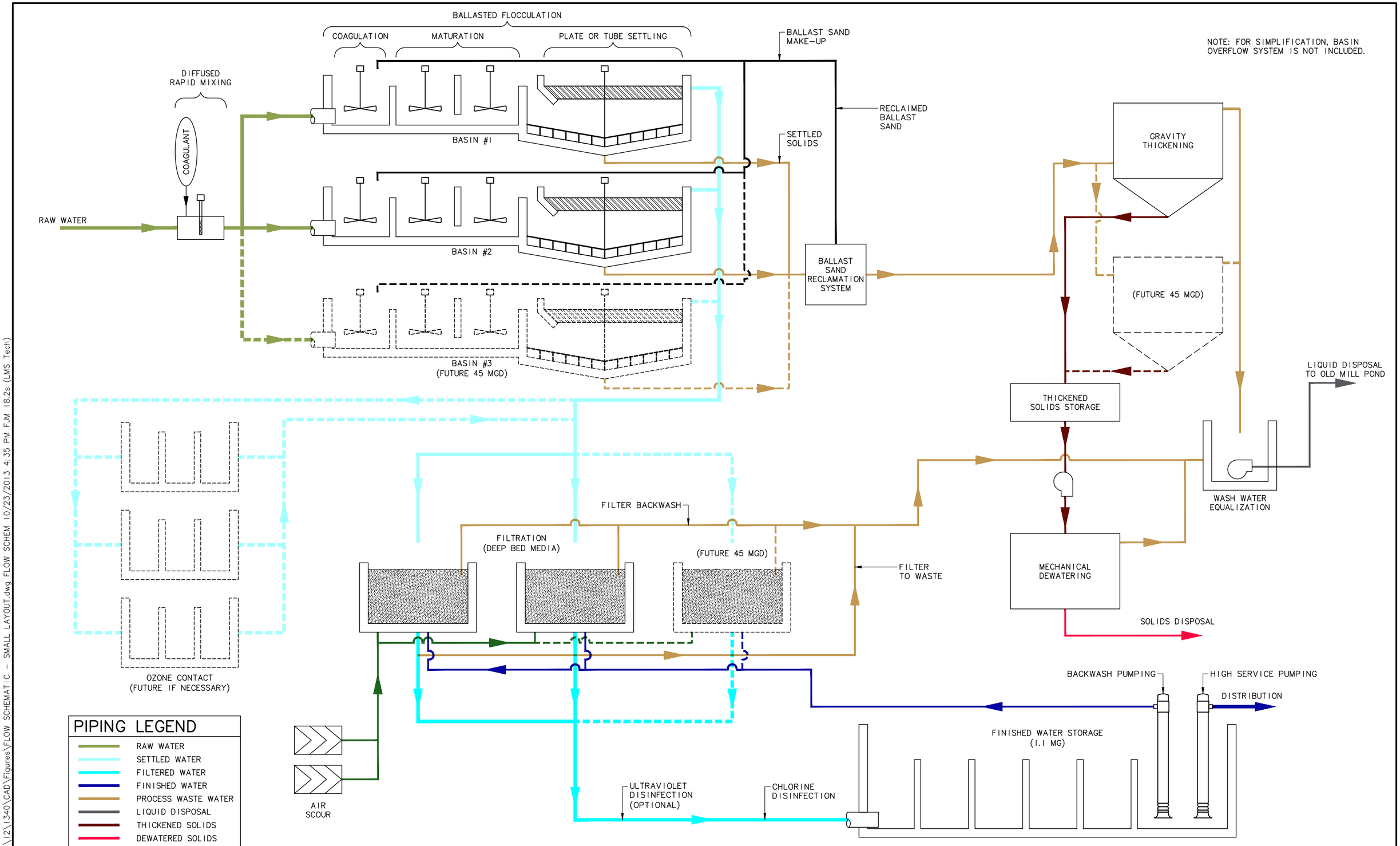
Alternative 5 differs from both Alternatives 3 and 4 in that it includes construction of washwater clarification basins.

Figures 8-1, 8-2, and 8-3 show process flow schematics for Alternatives 3, 4, and 5, respectively. Table 8-2 presents a comparison of the advantages and disadvantages for the three alternatives.

Facility Layouts and Construction Sequencing

Conceptual level site plans for Alternatives 3, 4, and 5 are shown in Figures 8-4, 8-5, and 8-6, respectively. The site plans shown for Alternatives 3 and 4 are intended to be representative of site layouts for each alternative without considering specific property or site orientation needs. It is expected that final site layouts would depend on the shape and orientation of the actual property. The site layout for Alternative 5 takes the unique dimensions and configuration of the City property into consideration. As the life expectancy of the new WTP structures would be expected to be a minimum of 75 years, the site plans include footprints associated with initial construction to achieve a capacity of 30 mgd and space provisions that allow expansion to an ultimate capacity of 45 mgd.

Based on the layouts, the property size requirements for a new WTP under Alternatives 3 and 4 ranges between 3.3 and 5.0 acres. These space requirements do not include additional space requirements that might become necessary for an irregularly-shaped parcel; unusable critical areas such as wetlands, steep slopes, or flood plains; unique land use codes or setbacks; or unfavorable geotechnical conditions. For the identified parcel under Alternative 5, all of the information known concerning such property constraints is considered.



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DRAWN F. MARESCALCO
CHECKED B. GINTER

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WATER TREATMENT PLANT FACILITY PLAN UPDATE
FIGURE 8-1
PROCESS FLOW SCHEMATIC - ALTERNATIVE 3

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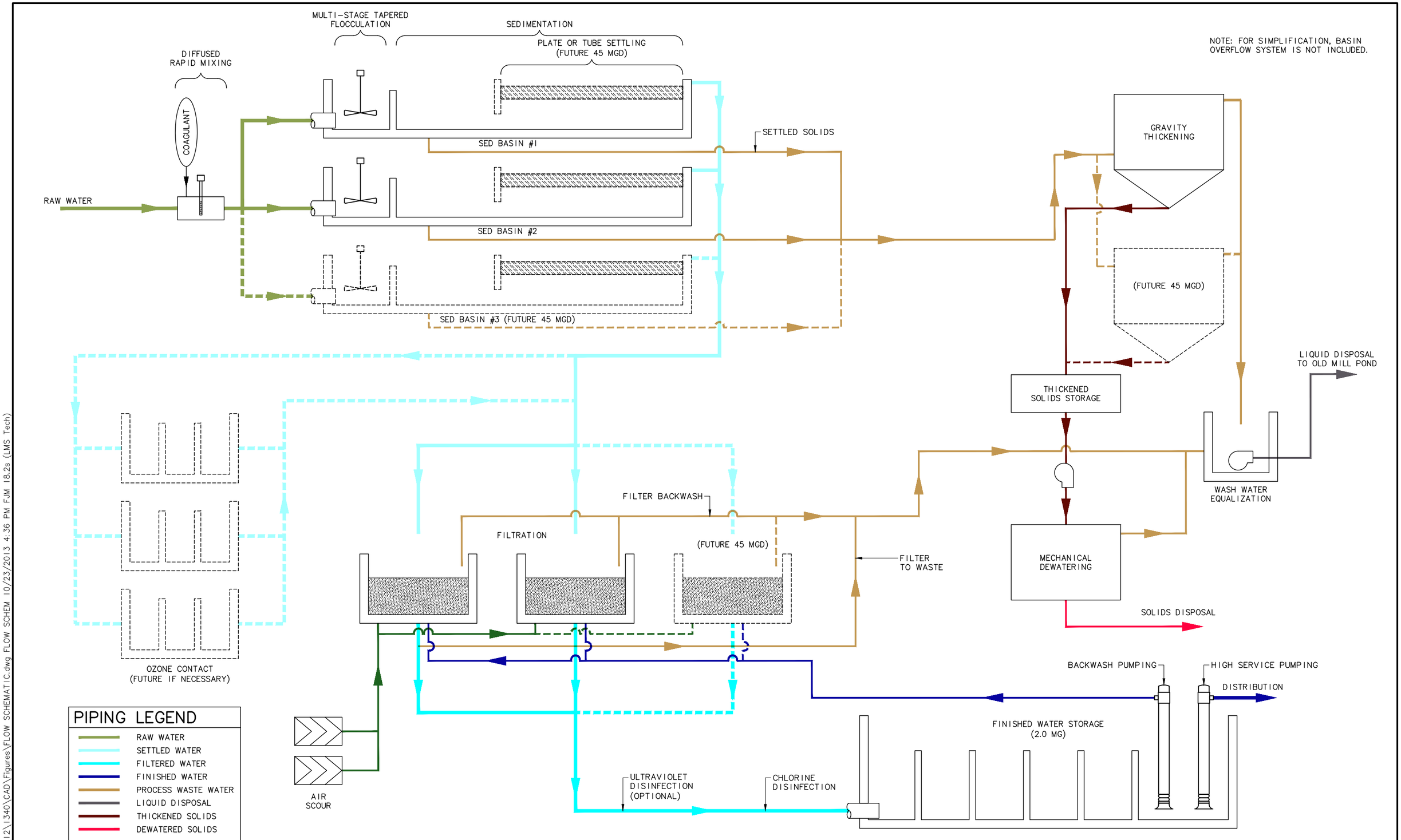

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WATER TREATMENT PLANT FACILITY PLAN UPDATE
FIGURE 8-2
PROCESS FLOW SCHEMATIC - ALTERNATIVE 4

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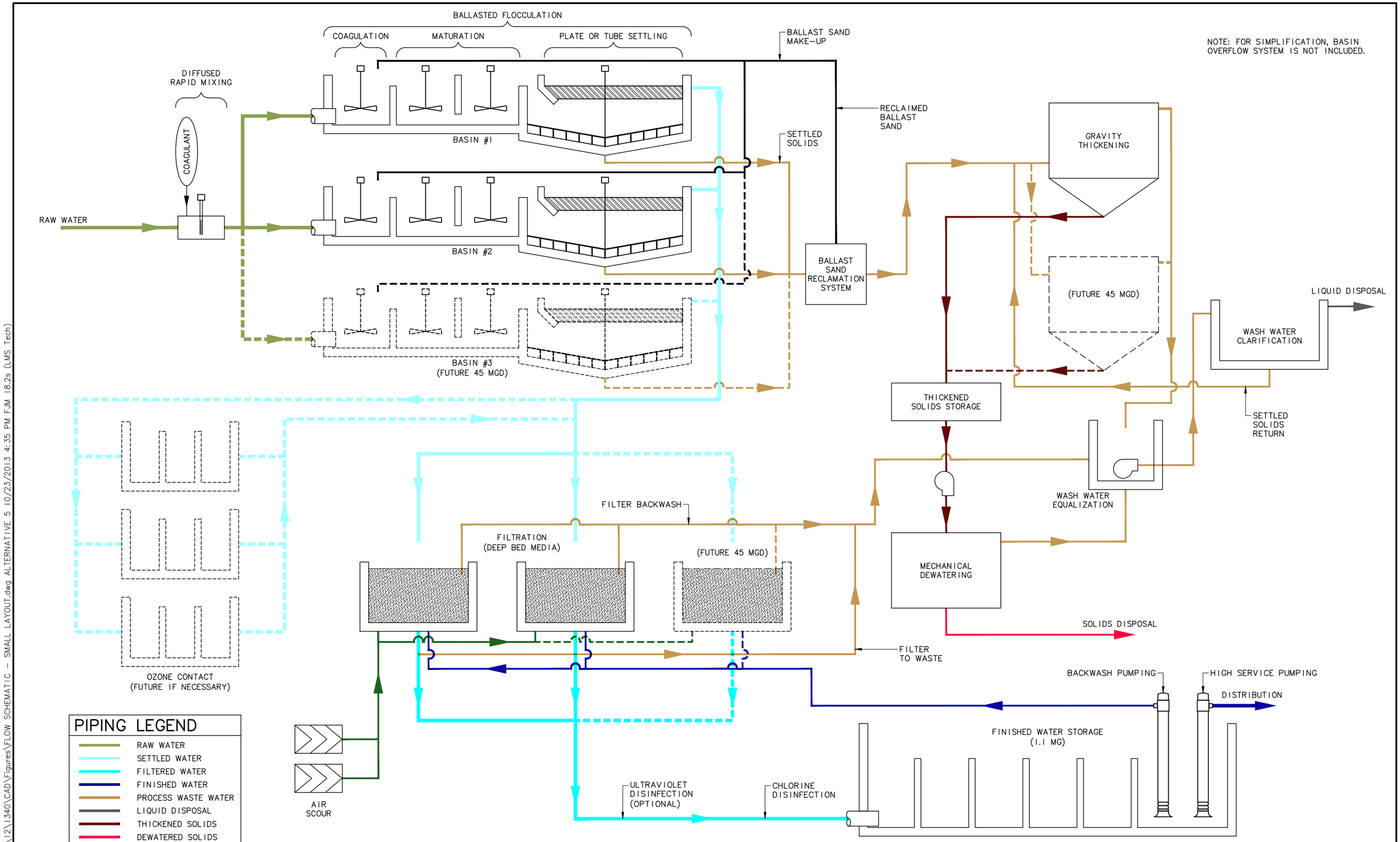

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WATER TREATMENT PLANT FACILITY PLAN UPDATE
FIGURE 8-3
PROCESS FLOW SCHEMATIC - ALTERNATIVE 5

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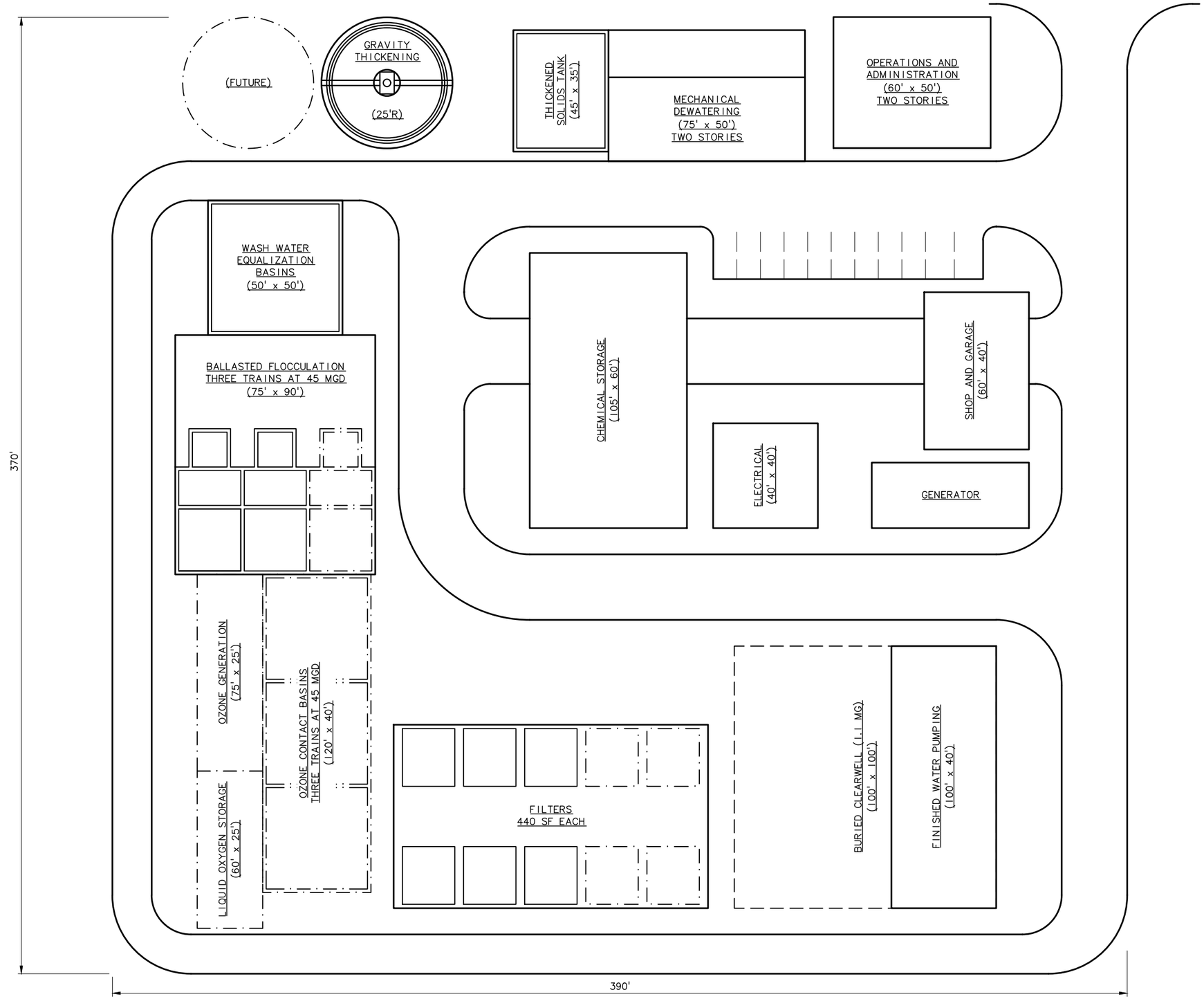

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CHECKED C. KELSEY

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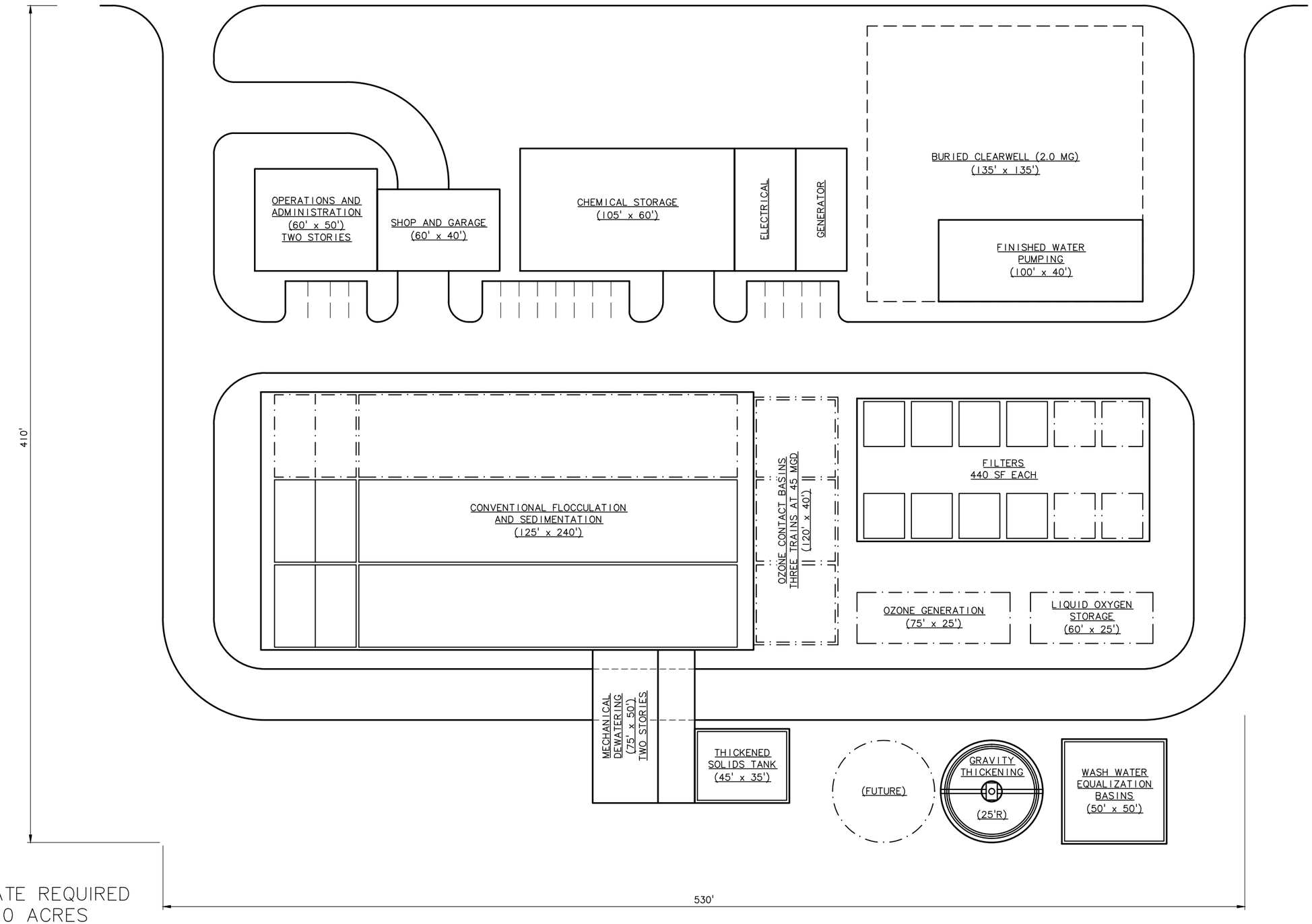
WATER TREATMENT PLANT FACILITY PLAN UPDATE
FIGURE 8-4
SITE LAYOUT - ALTERNATIVE 3



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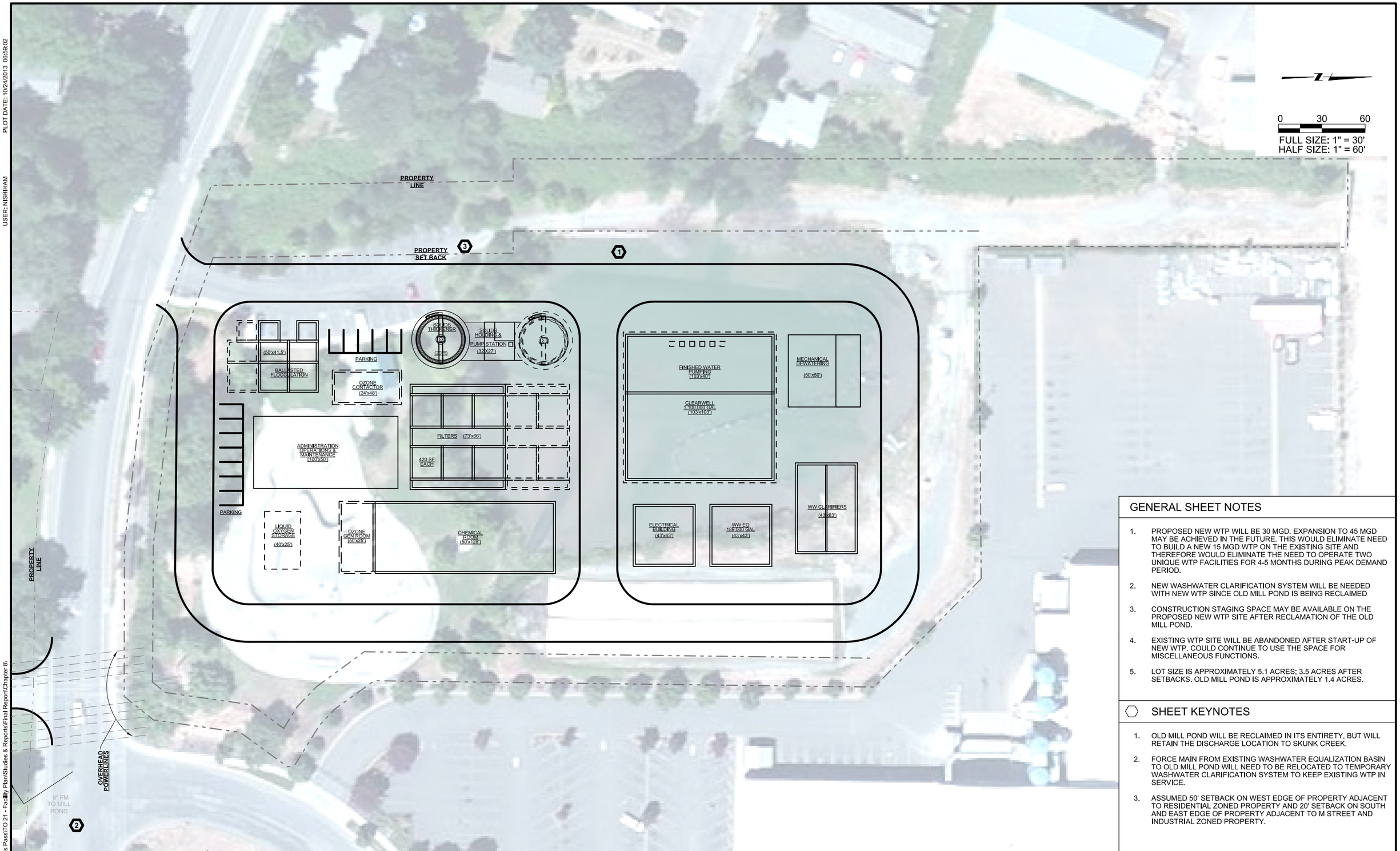
WATER TREATMENT PLANT FACILITY PLAN UPDATE
FIGURE 8-5
SITE LAYOUT - ALTERNATIVE 4



GENERAL SHEET NOTES

1. PROPOSED NEW WTP WILL BE 30 MGD. EXPANSION TO 45 MGD MAY BE ACHIEVED IN THE FUTURE. THIS WOULD ELIMINATE NEED TO BUILD A NEW 15 MGD WTP ON THE EXISTING SITE AND THEREFORE WOULD ELIMINATE THE NEED TO OPERATE TWO UNIQUE WTP FACILITIES FOR 4-5 MONTHS DURING PEAK DEMAND PERIOD.
2. NEW WASHWATER CLARIFICATION SYSTEM WILL BE NEEDED WITH NEW WTP SINCE OLD MILL POND IS BEING RECLAIMED
3. CONSTRUCTION STAGING SPACE MAY BE AVAILABLE ON THE PROPOSED NEW WTP SITE AFTER RECLAMATION OF THE OLD MILL POND.
4. EXISTING WTP SITE WILL BE ABANDONED AFTER START-UP OF NEW WTP. COULD CONTINUE TO USE THE SPACE FOR MISCELLANEOUS FUNCTIONS.
5. LOT SIZE IS APPROXIMATELY 5.1 ACRES; 3.5 ACRES AFTER SETBACKS. OLD MILL POND IS APPROXIMATELY 1.4 ACRES.

SHEET KEYNOTES

1. OLD MILL POND WILL BE RECLAIMED IN ITS ENTIRETY, BUT WILL RETAIN THE DISCHARGE LOCATION TO SKUNK CREEK.
2. FORCE MAIN FROM EXISTING WASHWATER EQUALIZATION BASIN TO OLD MILL POND WILL NEED TO BE RELOCATED TO TEMPORARY WASHWATER CLARIFICATION SYSTEM TO KEEP EXISTING WTP IN SERVICE.
3. ASSUMED 50' SETBACK ON WEST EDGE OF PROPERTY ADJACENT TO RESIDENTIAL ZONED PROPERTY AND 20' SETBACK ON SOUTH AND EAST EDGE OF PROPERTY ADJACENT TO M STREET AND INDUSTRIAL ZONED PROPERTY.

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SCALE: AS NOTED

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DRAWN A. NISHIHARA
CHECKED P. KREFT

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WATER TREATMENT PLANT FACILITY PLAN UPDATE
FIGURE 8-6
SITE LAYOUT - ALTERNATIVE 5

PAGE 8-14

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Table 8-2
Alternatives 3,4 and 5 Comparison Summary

Alternative	Advantages	Disadvantages
3	• Lowest initial construction and expansion costs • Smaller basin structures to maintain • Less property required	• Additional operator oversight of ballasted flocculation process • More mechanical systems to maintain • Additional regulatory approval may be required
4	• Proven clarification technologies for Grants Pass' Rogue River supply • Larger clearwell offers system storage reliability in addition to disinfection • Process retrofitting might offer capacity increases without new basin construction	• Requires more property • Higher initial construction and expansion costs
5	• City already owns the property • Smaller basin structures to maintain • Close to existing WTP and intake structure • Lower cost of connecting WTP to existing raw water and finished water pipelines.	• Geotechnical conditions of property are likely challenging • Permitting may be more difficult due to proximity to critical areas • Additional operator oversight of ballasted flocculation process • More mechanical systems to maintain • Additional regulatory approval may be required • Wetland mitigation and construction of a new skate park would be necessary

For each alternative, a single uninterrupted construction period of 28 months is estimated. Alternative 5 might require an increased duration for site preparation due to demolition and potential unsuitable soils. The construction period assumes that the contractor is allowed use of the entire undeveloped property for the duration of construction. Alternative 5 might require use of part of the existing WTP for staging and storage, if the City is willing to allow this. This assumption results in a shorter construction duration than those estimated for Alternatives 1 and 2 as presented in Chapter 7. The estimated construction duration for Alternatives 1 and 2 is approximately 48 to 54 months due to phasing of improvements.

Project Cost Estimates

Tables 8-3, 8-4, and 8-5 present planning-level project cost estimates for Alternatives 3, 4, and 5, respectively. The anticipated total project costs are expressed in 2013 dollars. These costs are for the initial construction under each alternative and will result in a new WTP with a rated capacity of 30 mgd. Tables 8-3, 8-4, and 8-5 do not show costs for expansion in 2065. A cursory review of City GIS property, land use, topography, and critical areas information suggests that there are several viable properties, in addition to the skate park property, within ½ mile of the existing plant. Costs associated with integrating a new WTP into the existing system were developed based on this general vicinity. Actual property acquisition and integration costs for Alternatives 3 and 4 will vary with site.

It is anticipated that construction of a new WTP would likely not begin for several years to allow time for property acquisition, design, environmental and regulatory permitting, public acceptance, financing, and bidding. Expansion to 45 mgd under any alternative would not take place until approximately 2065. Because the timing of capital outlays is different between alternatives, the net present value analysis of alternatives is presented in Chapter 9 for equivalent cost comparisons of all the alternatives.

Estimated project costs were developed using recent local industry information from estimates, bid tabs, vendor quotations, and other material unit costs for similar treatment facilities. Line item estimates represent installed costs that include materials, labor, equipment, and contractor overhead and profit.

These opinions of probable cost are based on planning-level analysis and a low level of project definition. These estimates are Class 5 estimates as defined in Chapter 7. They are subject to the following list of assumptions.

- No cost has been included for unusual site conditions requiring environmental remediation, poor soil conditions, or demolition of existing structures in Alternatives 3 or 4. Costs for extra foundations and remediation of poor soil conditions in Alternative 5 are based on similar projects. The cost for wetlands mitigation is based on an average cost of wetland mitigation banks in Oregon. Skate park construction costs are based on information associated with construction of the current park and appropriate escalation factors.
- No cost has been included associated with demolition of the existing WTP once the new plant is online.
- Cost for property acquisition in Alternatives 3 and 4 is based on a conservative assumption of recently assessed suitable properties in Grants Pass.
- Costs for piping connections to the existing raw water intake and the distribution system in Alternatives 3 and 4 are representative values and may vary widely depending on final site location. The cost for Alternative 5 is based on smaller assumed lengths because the location is known. All of the alternatives assume 48-

inch diameter steel pipe in public right-of-way. No cost for private easements is included.

- Costs for a permanent standby generator to produce approximately 5 MGD of finished water are included.
- No allowance is included for premium architectural finishes on plant structures. Concrete masonry unit construction with architectural metal roofing is assumed for building costs.
- Site civil and finishing costs will vary based on actual site size and layout.

Table 8-3
Alternative 3 Project Cost Estimate

Facility	Estimated Cost (2013 USD)
Mobilization and General Conditions (8 percent)	$2,400,000
Intake and Raw Water Pump Station Improvements	$1,450,000
Raw Water Transmission Main	$1,000,000
Rapid Mixing	$340,000
Clarification	$3,200,000
Filtration	$5,200,000
Treated Water Storage and Chlorine Contact Basin	$1,570,000
Finished Water Pumping and Metering	$4,400,000
Finished Water Transmission	$380,000
Process Wastewater Equalization Basin	$390,000
Backwash Force Main to Old Mill Pond	$400,000
Gravity Thickener	$1,500,000
Thickened Solids Storage Tank	$500,000
Mechanical Dewatering Structure and Equipment	$1,900,000
Chemical Storage and Feed Building and Equipment	$2,000,000
Maintenance, Operations, and Administration Building	$2,250,000
Site Electrical	$2,500,000
Miscellaneous Yard Piping	$260,000
Site Civil	$160,000
Site Finishing and Security	$80,000
Subtotal: Construction without Contingency	**$31,900,000**
Contingency (20 percent)	**$6,400,000**
Subtotal: Construction with Contingency	**$38,300,000**
Engineering, Permitting, Construction Management Services, Legal, Administration (25 percent)	**$8,000,000**
Property Acquisition	**$1,100,000**
Total Estimated Project Cost with Contingencies	**$47,400,000**

Table 8-4
Alternative 4 Project Cost Estimate

Facility	Estimated Cost (2013 USD)
Mobilization and General Conditions (8 percent)	$2,800,000
Intake and Raw Water Pump Station Improvements	$1,450,000
Raw Water Transmission Main	$1,260,000
Rapid Mixing	$340,000
Clarification	$4,500,000
Filtration	$7,500,000
Treated Water Storage and Chlorine Contact Basin	$2,630,000
Finished Water Pumping and Metering	$4,400,000
Finished Water Transmission	$380,000
Process Wastewater Equalization Basin	$390,000
Backwash Force Main to Old Mill Pond	$400,000
Gravity Thickener	$1,500,000
Thickened Solids Storage Tank	$500,000
Mechanical Dewatering Structure and Equipment	$1,900,000
Chemical Storage and Feed Building and Equipment	$2,000,000
Maintenance, Operations, and Administration Building	$2,225,000
Site Electrical	$2,500,000
Miscellaneous Yard Piping	$400,000
Site Civil	$240,000
Site Finishing and Security	$100,000
Subtotal: Construction without Contingency	**$37,400,000**
Contingency (20 percent)	**$7,500,000**
Subtotal: Construction with Contingency	**$44,900,000**
Engineering, Permitting, Construction Management Services, Legal, Administration (25 percent)	**$9,400,000**
Property Acquisition	**$1,100,000**
Total Estimated Project Cost with Contingencies	**$55,400,000**

Table 8-5
Alternative 5 Project Cost Estimate

Facility	Estimated Cost (2013 USD)
Mobilization and General Conditions (8 percent)	$2,700,000
Intake and Raw Water Pump Station Improvements	$1,450,000
Raw Water Transmission Main	$250,000
Rapid Mixing	$340,000
Clarification	$3,200,000
Filtration	$5,200,000
Treated Water Storage and Chlorine Contact Basin	$1,570,000
Finished Water Pumping and Metering	$4,400,000
Finished Water Transmission	$300,000
Process Wastewater Equalization Basin	$390,000
Washwater Clarification Basins	$600,000
Gravity Thickener	$1,500,000
Thickened Solids Storage Tank	$500,000
Mechanical Dewatering Structure and Equipment	$1,900,000
Chemical Storage and Feed Building and Equipment	$2,000,000
Maintenance, Operations, and Administration Building	$2,250,000
Site Electrical	$2,500,000
Miscellaneous Yard Piping	$260,000
Construction Dewatering	$700,000
Temporary Washwater Clarification Facilities	$250,000
Site Preparation	$1,000,000
Additional Cost for Building and Structure Pile Foundations	$3,000,000
Site Finishing and Security	$120,000
Subtotal: Construction without Contingency	**$36,400,000**
Contingency (20 percent)	**$7,300,000**
Subtotal: Construction with Contingency	**$43,700,000**
Engineering, Permitting, Construction Management Services, Legal, Administration (25 percent)	**$9,100,000**
Wetlands Mitigation Cost	**$600,000**
Property Acquisition and Skate Park Construction	**$800,000**
Total Estimated Project Cost with Contingencies	**$54,200,000**

A new WTP constructed under any new WTP alternative would require expansion from a capacity of 30 mgd to a capacity of 45 mgd in approximately 2065. Expansion to 45 mgd under any alternative requires the construction of a new intake structure and raw water pump station and the construction of additional treatment trains. Estimated project costs for this

expansion for each alternative are shown in Table 8-6. These expansion costs affect the net present value of alternatives which are developed and presented in Chapter 9.

Table 8-6
Estimated Project Cost for Plant Expansions in 2065

Alternative	Estimated Project Cost (2013 USD)
3	$33,000,000
4	$36,700,000
5	$37,000,000

Note: This is a Class 5 estimate. The accuracy ranges from –30 percent to +50 percent.

Near-Term Improvements

During construction of a new WTP, the existing plant would continue to supply drinking water to the system. The structural condition of the clearwell at the existing WTP is of such concern that the team investigated a separate project which would increase short-term disinfection and supply reliability at the existing WTP. A project cost of approximately $450,000 was developed based on a combination of structural fortification within the clearwell and plumbing provisions to allow emergency insertion of post-filtration UV disinfection units if the clearwell were to fail. This project cost was included in economic calculations for new treatment plant alternatives in order to provide a conservative financial comparison to other alternatives.

Based on further investigation of the feasibility of completing such improvements in the clearwell, including recent analysis of potential structural improvements, it was concluded that such improvements cannot be completed while maintaining adequate water production to meet the City's water demands. Because of these difficulties and the fact that no investment can effectively mitigate damage in a major event, the project was not investigated any further.

Summary

Alternatives 3, 4, and 5 propose constructing a new WTP on a new site and each have a range of initial capital costs and operational implications, similar to those presented in Chapter 7. The alternatives also offer differing approaches to layout and configuration, each with varying advantages, disadvantages, and estimated project costs. They also define a range of required property size for the purposes of selecting an appropriate location. A comparative evaluation of all five alternatives, which includes social and environmental considerations in addition to the costs developed, is presented in Chapter 9 and is used as the basis for capital improvement recommendations.

CHAPTER 9
CAPITAL IMPROVEMENT PROGRAM RECOMMENDATIONS

Introduction

Over the past decade, evaluation of alternative utility engineering solutions through the use of TBL evaluations has become commonplace. TBL decision-making is a process where evaluations consider social and environmental impacts in addition to the economic aspects of a proposed project.

Within the water and wastewater industries, TBL evaluations have been employed for projects where the capital investment and anticipated longevity of constructed facilities have long-term impacts to the image and culture of a community. Treatment facilities are the most common types of projects where TBL evaluations have been used. By including community leaders during the TBL process for evaluating alternative improvements, a measure of public involvement and consensus building can be achieved. The recommended project solution then becomes more reflective of the community's culture.

Due to the importance of this CIP, an Advisory Committee of community leaders and City Council (Council) members was assembled to assist in the evaluation and recommendation of a preferred alternative from those presented in Chapters 7 and 8. City Public Works employees integral to the project also participated to offer input on operational impacts, zoning and land use information, and necessary steps in the City approval process.

A series of four workshops were conducted over a three-month period with the Advisory Committee using an independent facilitator. The MSA Team's role in these workshops was to present information on the alternatives developed and to answer technical questions posed by committee members. Below is a summary of activities for each of the four workshops:

- Workshop 1 (May 14, 2013): Introduction of Advisory Committee members, consulting team, and public works employees; tour of the existing WTP; and dissemination of suggested TBL evaluation criteria. Draft text for Chapters 1 through 8 of this Facilities Plan Update was also made available for review.
- Workshop 2 (May 30, 2013): Discussion and finalization of TBL criteria and individual weighting; presentation and questions-and-answers period for each of the capital improvement alternatives; distribution of TBL scoring matrix spreadsheets to committee members for review and scoring.
- Workshop 3 (June 4, 2013): Review of information requested by the committee concerning alternative property constraints (setbacks and relocation of overhead power lines); discussion and scoring of alternatives; request for development of a fifth alternative.
- Workshop 4 (July 15, 2013): Presentation of requested Alternative 5, finalization of committee scoring, and development of recommendation to Council.

The following sections discuss the development of TBL evaluation criteria, considerations included within each evaluation category, scoring of alternatives, and the CIP recommendation. The final sections of the chapter outline an implementation plan for the recommended program.

Development and Weighting of TBL Criteria

For the benefit of the Advisory Committee, a list of suggested criteria for each of the TBL categories was developed from similar projects. The committee then modified and finalized the criteria, establishing appropriate weighting for each through group discussion. Each criterion was assigned a weighting from 1 to 5, with 5 representing the highest level of importance. The final criteria and weightings are offered in Table 9-1. Definitions for the economic, social, and environmental categories are discussed in the following sections.

Economic Measures

Economic variables are those that deal with the flow of money or change in financial value. These factors consider income or expenditures, taxes, business climate factors, and employment. A net present value analysis was performed by the MSA team and presented to the Advisory Committee during the workshops. The net present value analysis is summarized later in this chapter.

Social Measures

Social variables include measurements of education, equity, access to resources, health and well-being, quality of life, and social capital. The social variables identified by the Advisory Committee as most important for the selection of an alternative are described below.

Safe water supply

A safe water supply is one that is free of pathogens and microorganisms that, if ingested, can cause mild to severe illness and even death. In addition to the absence of pathogens, a safe water supply should be free of cancer-causing toxins such as heavy metals, pesticides, herbicides, and solvents.

Reliable Water Supply

Western Oregon borders the Cascadia subduction zone. A comprehensive study lead by researchers at Oregon State University published by the USGS in 2012 predicted that if an earthquake were to occur along the Cascadia fault, it would have a magnitude between 8.7 and 9.2 as calculated by the Richter magnitude scale. Buildings and infrastructure not up to current seismic code could be compromised or completely destroyed in the event of such a large earthquake. Grants Pass has only one source of drinking water. The reliability of the water supply during emergencies such as a fire, earthquake, or drought is critical to the

community. Structures, mechanical equipment, and electrical infrastructure supporting the WTP need to be reliable to ensure that water is available whenever it is needed.

Table 9-1
Scoring Criteria and Weighting Summary

Criteria		Weighting
	Economic Measures	
1	Capital cost	5
2	Operations and maintenance costs	5
3	Net present value	5
4	Rate impact	2
5	Sustaining existing industry	3
6	Job growth opportunities	4
7	Construction period impacts	1
	Economic Measures Weighting Subtotal	25
	Social Measures	
8	Safe water supply	4
9	Reliable water supply	4
10	Community growth	3
11	Operability and staff accommodations	1
12	Construction impact	2
13	Historical values	4
	Social Measures Weighting Subtotal	18
	Environmental Measures	
14	Proximity of new facilities to existing intake	1
15	Energy efficiency of structures	5
16	Solids handling	4
17	Electricity consumption	3
18	Change in land use	5
19	Construction period impacts	2
	Environmental Measures Weighting Subtotal	20
	Total Weighted score	**63**

Community Growth

Last year, MSA did a planning study for the City of Grants Pass and projected that the population will grow to 90,173 people by the year 2065. This increase in population will lead to increased demand for potable water. In addition, potential new industrial and commercial development that comes with population growth can further increase water demand.

Operability and Staff Accommodations

A good WTP design will allow for safe, efficient, straightforward operation by plant staff. Design elements such as the use of guard rails, automated pumps, leak detection systems, color coding, telemetry, and the elimination of tripping hazards are just a few examples of the many considerations engineers and contractors make to ensure operability for a safe community asset.

Construction Impact

Construction activities can have a significant impact on residents' quality of life. During construction, residents may be exposed to loud noises, trucks and heavy machinery driving through their neighborhoods, a temporary decline in air quality, and the potential for water service interruptions which may be accidental or necessary for certain phases of construction.

Historical Values

The City of Grants Pass is currently served by the second-oldest water treatment plant in the State of Oregon. The plant was designated an American Water Landmark by AWWA in 1998 and holds nostalgic value for many residents.

Environmental Measures

Environmental variables consider natural resources and the potential impacts a project may have on them. Some factors include air and water quality, energy consumption, natural resources, solid and toxic waste, and land use. Brief descriptions of the environmental variables that the Committee considered during its analysis are presented below.

Proximity of New Facilities to Existing Intake

The closer a WTP is to the location of its intake, the less the surrounding environment is impacted. Water must be conveyed to the WTP from the intake via large-diameter piping, which can be challenging to install without significant environmental impacts.

Energy Efficiency of Structures

Energy-efficient structures have several environmental benefits. These benefits include, but are not limited to, minimizing air pollution, reducing carbon footprint, decreasing thermal pollution, and reducing greenhouse gas emissions.

Solids Handling

Water treatment plant solids consist primarily of silts, sands, and organics that are transported with the river water through the intake and either settled out or filtered out at the WTP. A good solids handling plan can have many environmental benefits, but a poor solids

handling plan can result in negative environmental impacts. There are many things to consider when developing a plan for solids handling including the presence of treatment chemicals, off-hauling, facility footprint, and energy usage.

Electricity Consumption

The way a WTP is designed and operated has far-reaching impacts to the plant's consumption of electrical power. New structures and treatment processes normally offer more energy conservation potential than retrofitting existing processes. Hydraulic conditions, plant location, distribution system design, valves, automated controls, and timing of production are just a few of the aspects of a WTP that affect energy consumption.

Change in Land Use

Choosing an alternative that requires a new WTP site would likely require a change in the land use designation at the new site. It is better to locate a new WTP so that it does not disrupt habitat for wildlife.

Construction Period Impacts

During construction, workers and the environment are at a heightened risk of impact. There may be exposure to toxic fumes; soil contamination; excessive runoff into surrounding surface water bodies; disturbance of lead-based paint, caulk containing PCBs, or asbestos; and inadvertent spills of asphalt or chemicals. Sound construction practices can reduce these risks and the risks are different between working on an existing structure and building a new structure.

Evaluation of Alternatives

This section describes how each alternative was evaluated against the criteria defined by the Advisory Committee. A net present value analysis was used to evaluate each alternative's economic aspects. The detailed analyses of each alternative with respect to social and environmental considerations were performed by the Advisory Committee during the course of its workshops.

Economic Considerations

A net present value analysis was performed to compare alternatives on the basis of cost. The net present value is a better way to compare costs between the alternatives than comparing the project costs developed in Chapters 7 and 8 because each alternative proposes the expenditure of different amounts of money at different times. In the present value analysis, each expenditure is escalated to the anticipated year of occurrence and then discounted back to a common year. In this analysis, the common year is 2013.

The Baseline Alternative, briefly described in Chapter 6, proposes to make approximately $12.5 million in structural upgrades to the existing WTP structures. This alternative was not included in the detailed economic study for the following reasons:

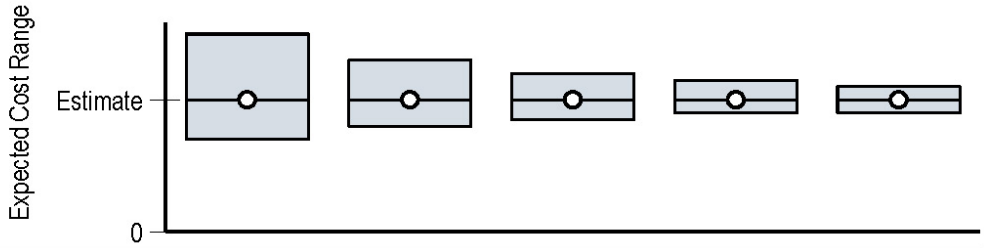
- Some of the structures that would initially be renovated would be demolished during later improvements needed to increase plant capacity. A significant portion of the investment to renovate those structures would be wasted.
- The existing plant would still operate inefficiently, so annual operations and maintenance costs would continue to be higher than other alternatives.
- This alternative does not address short-term capacity needs, long-term capacity needs, or structural longevity needs beyond year 2065. A new WTP with a capacity of 45 mgd would need to be built in a new location in 2065. The approximate cost of this new WTP would be $75.4 million (2013 dollars).

The Baseline Alternative was included during the workshops, however, and the Advisory Committee's analysis confirmed that this alternative is not a desirable solution.

Project Definition Level and Cost Index

The American Association of Cost Engineers (AACE) defines classes of cost estimating based on the level of project definition. The accuracy of cost estimates varies with the level of project definition. As shown in Figure 9-1, estimating accuracy improves as project definition increases.

Figure 9-1
Cost Estimating Accuracy Based on Level of Project Definition



Estimate Class	Class 5	Class 4	Class 3	Class 2	Class 2
Level of Project Definition	1%	10%	30%	60%	90%
Accuracy Range	-30%/+50%	-20%/+30%	-15%/+20%	-10%/+15%	-10%/+10%

Adapted from AACE International Recommended Practice No. 18R-97

AACE considers the type of planning work done for this Facility Plan Update to be a very low level of project definition, corresponding to somewhere between 1 and 3 percent complete. It is likely that changes in the construction market or overall economy, new regulatory requirements, site conditions, and other factors will affect the total project cost.

The costs prepared for this Facility Plan Update are subject to the accuracy range of –30 percent to +50 percent as shown in Figure 9-1.

Construction costs are also subject to change with time. All of the costs used in this chapter are in 2013 dollars. It will be necessary to adjust these present cost estimates in the future. An indexing method is useful for this purpose. The Engineering News Record (ENR) Construction Cost Index (CCI) is a commonly used index for this purpose. For purposes of future cost estimate updating, the December 2013 ENR CCI for Seattle, Washington is 10142.65.

Capital Costs

For the present value analysis, an escalation rate of 2 percent was assumed. A discount rate of 3 percent was used. These parameters are used to predict the effects of deferring project costs. The escalation rate is a measure of the general fall in the purchasing value of money, also called inflation. The discount rate reflects the value to the City in deferring capital costs. The analysis is carried through 2095 which is consistent with the planning period identified in Chapter 6.

Alternatives 1 and 2

Alternatives 1 and 2 propose improvements to existing plant structures and processes as discussed in Chapter 7. These improvements must occur in separate phases because the plant must remain online during construction. The capital costs for these improvements were shown in Chapter 7 and each line item was assigned to a specific project phase. To develop phase costs, the capital costs for items associated with each phase were added together and associated project costs were distributed proportionally. Phases are anticipated to occur three years apart and begin in year 2018. As discussed in Chapter 7, Alternative 1 requires the construction of a new 45-mgd plant in 2065 and Alternative 2 requires the construction of a new 15-mgd plant in 2065. Tables 9-2 and 9-3 show the net present value for project costs associated with Alternative 1 and Alternative 2, respectively.

Table 9-2
Alternative 1 Project Cost Present Value Summary

Description	Current Cost	Year Spent	Escalated Cost	Present Value
Phase A	$9,000,000	2018	$9,936,727	$8,571,508
Phase B	$12,700,000	2021	$14,880,074	$11,746,468
Phase C	$15,700,000	2024	$19,520,977	$14,102,369
New 45-mgd Plant Construction	$75,400,000	2065	$211,144,745	$45,398,823
			Net Present Value	**$79,819,168**

Note: This is a Class 5 estimate. The accuracy ranges from –30 percent to +50 percent.

Table 9-3
Alternative 2 Project Cost Present Value Summary

Description	Current Cost	Year Spent	Escalated Cost	Present Value
Phase A	$12,300,000	2018	$13,580,194	$11,714,395
Phase B	$27,200,000	2021	$31,869,135	$25,157,790
Phase C	$17,300,000	2024	$21,510,376	$15,539,553
New 15-mgd Plant Construction	$47,202,000	2065	$132,181,091	$28,420,627
			Net Present Value	**$80,832,364**

Note: This is a Class 5 estimate. The accuracy ranges from –30 percent to +50 percent.

Alternatives 3, 4, and 5

Alternatives 3, 4, and 5 propose construction of a new WTP at a new site as discussed in Chapter 8. If any alternative were to be implemented, the City may need to construct immediate improvements at the existing WTP to ensure disinfection reliability while planning and building the new facilities. The construction of a new WTP is not phased like improvements to the existing plant are, but an expansion will be required in 2065 under any alternative to increase plant capacity from 30 mgd to 45 mgd. The need for property acquisition and environmental studies and permitting is anticipated to delay completion of a new WTP to the year 2020. Tables 9-4, 9-5, and 9-6 show the net present value for project costs associated with Alternatives 3, 4, and 5, respectively.

Table 9-4
Alternative 3 Project Cost Present Value Summary

Description	Current Cost	Year Spent	Escalated Cost	Present Value
Near-Term Disinfection Reliability	$450,000	2013	-	$450,000
Initial Construction to 30 mgd	$47,400,000	2020	$54,447,701	$44,270,963
Expansion of WTP to 45 mgd	$32,956,000	2065	$92,287,616	$19,843,019
			Net Present Value	**$64,563,982**

Note: This is a Class 5 estimate. The accuracy ranges from –30 percent to +50 percent.

Table 9-5
Alternative 4 Project Cost Present Value Summary

Description	Current Cost	Year Spent	Escalated Cost	Present Value
Near-Term Disinfection Reliability	$450,000	2013	-	$450,000
Initial Construction to 30 mgd	$55,400,000	2020	$63,637,186	$51,742,856
Expansion of WTP to 45 mgd	$36,668,000	2065	$102,682,434	$22,078,038
			Net Present Value	**$74,270,893**

Note: This is a Class 5 estimate. The accuracy ranges from –30 percent to +50 percent.

Table 9-6
Alternative 5 Project Cost Present Value Summary

Description	Current Cost	Year Spent	Escalated Cost	Present Value
Near-Term Disinfection Reliability	$450,000	2013	-	$450,000
Initial Construction to 30 mgd	$54,200,000	2020	$62,258,763	$50,622,072
Expansion of WTP to 45 mgd	$36,990,000	2065	$103,584,140	$22,271,916
			Net Present Value	**$73,343,988**

Note: This is a Class 5 estimate. The accuracy ranges from –30 percent to +50 percent.

Operations and Maintenance Costs

All annual costs are projected based on recent existing plant operating cost records and are increased proportional to projected demand increases and escalation rates. At this planning level, no difference in annual operational costs can be justified between alternatives 1, 3, 4, and 5 even though some technologies might require slightly more power, slightly less chemical usage, or some other subtle difference. Alternative 2 has higher labor costs starting in 2065, when two separate treatment plants would begin to operate. Table 9-7 shows the assumed annual 2013 value for each operating cost category and the lump sum of each operating cost over the entire 75-year planning period in 2013 dollars.

Summary of Net Present Value Analysis

The total net present value of each project is the sum of the annual costs and the capital costs, discounted back to the same year. Table 9-8 shows a summary of all of the alternatives with the complete lifecycle cost in present value. According to the analysis, building a new WTP has a lower lifecycle cost than upgrading the existing WTP.

Table 9-7
Operations and Maintenance Costs Present Value Summary[1]

Description	Annual Cost in 2013 US Dollars	Total Present Value[2]
Power	$287,873	$32,368,433
Labor	$601,280	$61,111,423
Chemicals	$176,097	$19,759,248
General Maintenance and Equipment Recovery	$339,915	$34,161,690
Net Present Value, Alternatives 1, 3, 4, and 5		**$147,400,793**
Additional Cost to Manage Two Plants[3]	$300,640	$11,992,689
Net Present Value, Alternative 2		**$159,393,482**

Notes
1. Values are scaled annually according to increases in production and general inflation
2. Lump sum of all annual payments made over 75-year planning period
3. Alternative 2 only
4. This is a Class 5 estimate. The accuracy ranges from –30 percent to +50 percent.

Sensitivity to Economic Conditions

The present value analysis shows which alternative has the lowest overall lifecycle cost in 2013 dollars. The analysis relies on planning criteria established in Chapter 6 and assumptions which are representative of normal industry and economic conditions. It is possible that these conditions could change. Therefore, the sensitivity of the analysis was investigated by modifying parameters which reflect economic conditions, demand projections, and assumptions about risk associated with construction at the existing WTP. This section presents a summary of these analyses and their effects on the lifecycle costs of the five alternatives.

The escalation and discount rates used in the base present value analysis are 2 percent and 3 percent, respectively. These parameters are representative of the economic climate of the past several decades. In a robust economy, the difference between the escalation and discount rates would be larger. In a more depressed economy, the difference would be smaller. In order to simulate these two types of economies, the present value analysis was repeated. To represent a robust economy, an escalation rate of 2 percent and discount rate of 5 percent were selected. In the depressed economy scenario, the escalation rate is 2.8 percent and the discount rate is 3 percent. Table 9-9 shows how these different economic conditions affect the results of the present value analysis.

The results of the sensitivity analysis indicate that in an unusually robust economy, Alternatives 1 and 3 are comparable in lifecycle cost over the planning period. In any other situation, Alternative 3 has the lowest lifecycle cost. In a typical or depressed economy, Alternative 3 has a lower lifecycle cost than the other alternatives, and building a new WTP costs less than upgrading the existing WTP.

Table 9-8
Net Present Value Analysis Summary

Item Description	Alternative 1	Alternative 2	Alternative 3	Alternative 4	Alternative 5
Capital Costs					
Phase A	$8,572,000	$11,715,000	-	-	-
Phase B	$11,747,000	$25,158,000	-	-	-
Phase C	$14,103,000	$15,540,000	-	-	-
New 15 MGD Plant Construction	-	$28,421,000	-	-	-
New 45 MGD Plant Construction	$45,400,000	-	-	-	-
New 30 MGD Plant Construction	-	-	$44,271,000	$51,743,000	$50,622,000
Expansion to 45 MGD	-	-	$19,844,000	$22,079,000	$22,272,000
Near-Term Disinfection Reliability	-	-	$450,000	$450,000	$450,000
Annual Costs					
Power	$32,369,000	$32,369,000	$32,369,000	$32,369,000	$32,369,000
Labor	$61,112,000	$61,112,000	$61,112,000	$61,112,000	$61,112,000
Chemicals	$19,760,000	$19,760,000	$19,760,000	$19,760,000	$19,760,000
General Maintenance and Equipment Recovery	$34,162,000	$34,162,000	$34,162,000	$34,162,000	$34,162,000
Additional Cost to Manage Two Plants	-	$11,993,000	-	-	-
Total Present Value	**$227,200,000**	**$240,200,000**	**$212,000,000**	**$221,700,000**	**$220,700,000**

Note: This is a Class 5 estimate. The accuracy ranges from –30 percent to +50 percent.

Table 9-9
Economic Sensitivity of Present Value Analysis Summary

Alternative	1	2	3	4	5
Robust Economy, Escalation Rate = 2 percent, Discount Rate = 5 percent					
Capital Cost Present Value	$45,973,000	$55,245,000	$46,445,000	$53,798,000	$52,889,000
Annual Cost Present Value	$67,983,000	$71,155,000	$67,983,000	$67,983,000	$67,983,000
Rounded Total Present Value	**$114,000,000**	**$126,400,000**	**$114,400,000**	**$121,800,000**	**$120,900,000**
Depressed Economy, Escalation Rate = 2.8 percent, Discount Rate = 3 percent					
Capital Cost Present Value	$104,938,000	$98,563,000	$76,998,000	$88,246,000	$87,352,000
Annual Cost Present Value	$213,660,000	$234,409,000	$213,660,000	$213,660,000	$213,660,000
Rounded Total Present Value	**$318,600,000**	**$332,900,000**	**$290,700,000**	**$301,900,000**	**$301,000,000**

Note: This is a Class 5 estimate. The accuracy ranges from –30 percent to +50 percent.

Construction Risk at Existing Water Treatment Plant

In Chapter 7, the risk of constructing improvements at the existing WTP was discussed. These risks are due to unpredictable construction conditions at the WTP and the difficulty associated with making the improvements while keeping the existing WTP on-line. The cost estimates for Alternatives 1 and 2 accounted for those risks by incorporating additional costs using methodologies explained in Chapter 7. These added costs influence the lifecycle cost of Alternatives 1 and 2. To examine the effects of those assumptions on the present value analysis, the analysis was repeated without those added costs. This analysis was done under the base economic conditions of 2 percent escalation and 3 percent discount. The results of analysis without addition of any risk to Alternatives 1 and 2 are summarized in Table 9-10.

Table 9-10
Present Value Analysis Results with No Additional Risk at Existing WTP

Alternative	1	2	3	4	5
Capital Cost Present Value	$74,945,000	$72,437,000	$64,565,000	$74,272,000	$73,344,000
Annual Cost Present Value	$147,403,000	$159,396,000	$147,403,000	$147,403,000	$147,403,000
Rounded Total Present Value	**$222,300,000**	**$231,800,000**	**$212,000,000**	**$221,700,000**	**$220,700,000**

Note: This is a Class 5 estimate. The accuracy ranges from –30 percent to +50 percent.

If there is no additional risk associated with construction of improvements at the existing WTP, the initial capital costs of Alternatives 1 and 2 are lower as expected. However, Alternatives 3 and 4 have lower lifecycle costs than Alternatives 1 and 2, even under these unlikely assumptions.

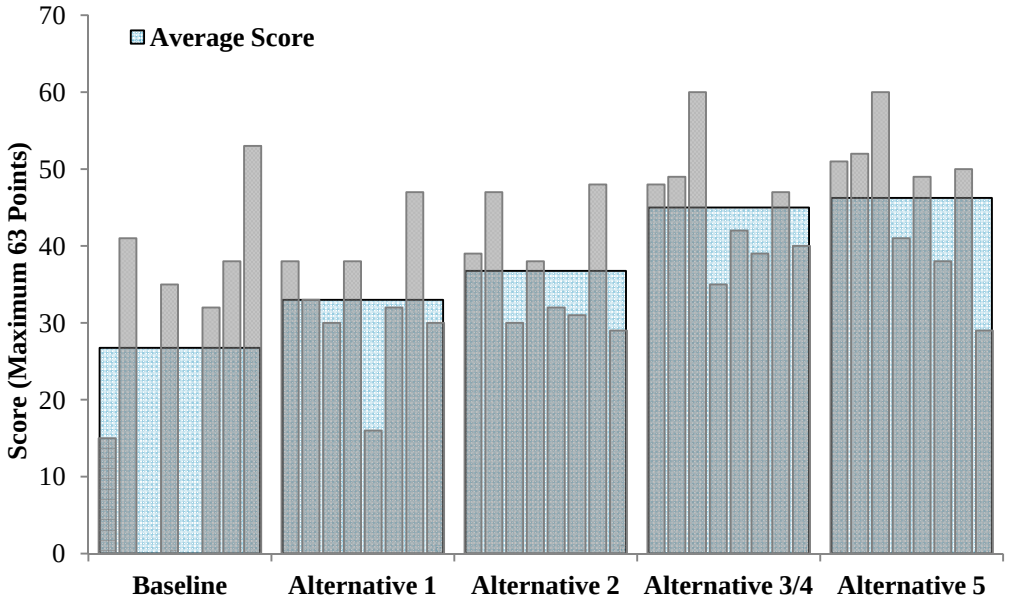
Social and Environmental Considerations

The net present value analysis used to compare the economic aspects of each alternative is considered an objective process because it relies on quantities and calculations. The social and environmental analyses of the alternatives are subjective processes because they cannot be easily quantified and are subject to interpretations based on opinions. Therefore, the detailed analyses with respect to social and environmental considerations were left to the members of the Advisory Committee. The details of the social and environmental analyses are beyond the scope of this report. The results are presented below.

Alternative Selection and Recommendation

Following presentations and discussions of each Alternative, the members of the Advisory Committee independently scored the alternatives by each of the established TBL criteria. Scores were given on a scale of 1 to 5, with a score of 5 meaning that the alternative in question was considered the most desirable with respect to the given criterion. The scores assigned were scaled and multiplied by each criterion's weighting factor to derive individual and composite scores for each alternative. Alternatives 3 and 4 were scored collectively as a composite alternative representing a new WTP on an undefined property. The individual and composite scores are summarized in Figure 9-2.

Figure 9-2
Advisory Committee Alternative Scoring Results



Workshop results and scoring were presented to the City Council during its August 5, 2013 Workshop and then discussed further during its September 9, 2013 workshop. In reviewing the materials developed and scoring performed, the Council approved completion of this Facility Plan Update with the recommendation to move forward in the planning process to construct a new WTP. Due to the close scoring between Alternatives 3, 4, and 5, the Council instructed that a detailed investigation of prospective properties be conducted to identify the optimal site from a cost, facility layout, permitting, and constructability standpoint.

Capital Improvement Program Implementation Plan

As detailed in Chapter 8, the conceptual project cost to construct a new WTP is estimated to be approximately $56 million, with an accuracy range of -30% to +50% (Class 5 estimate). It is recommended that the City establish a capital budget for this project which reflects this estimate and the level of uncertainty and risk associated with the current level of project definition. This budget should be updated and refined over time as the implementation plan progresses and planning and design uncertainties are addressed.. The budget should include decommissioning and demolition of the existing WTP. If pilot testing and other near-term activities demonstrate the ability to use higher-rate treatment processes which require less space and have lower construction costs, such as Alternative 3, then it may be possible to reduce the total project expenditures accordingly.

The recommended schedule to implement the new WTP is presented in Figure 9-3. It is possible to have a new WTP online by the middle 2019 using a traditional design-bid-build (DBB) project delivery approach.

The City should implement the new WTP as quickly as possible to avoid extensive investments in the existing plant. Keeping the current WTP online presents structural and seismic risks and risks related to other deficiencies. The required capital investment in the existing WTP to mitigate these risks will increase as time goes on. The Advisory Committee was not tasked with addressing this schedule-related risk challenge, but the City staff and City Council should discuss this topic as part of its planning and budgeting process for fiscal year 2014-2015 and beyond.

It is recommended that the City complete an Emergency Response Plan for the existing WTP and related water supply infrastructure to allow the City to make informed decisions related to the risks at the existing WTP. This Emergency Response Plan is the minimum investment that the City should make while it waits to have a new WTP designed and constructed. This planning work may identify additional investments needed to mitigate for risks that cannot be effectively managed.

Table 9-11 presents a summary of anticipated yearly capital expenditures (project costs) for the next 10 years to implement a new WTP based on the Implementation Schedule discussed above. The considerations and recommended tasks to undertake a project of this magnitude are presented in the following sections. There are no capital investments required for the City's water treatment and supply system in the next 10 years after the new WTP becomes operational in 2019, so the CIP planning horizon is 10 years.

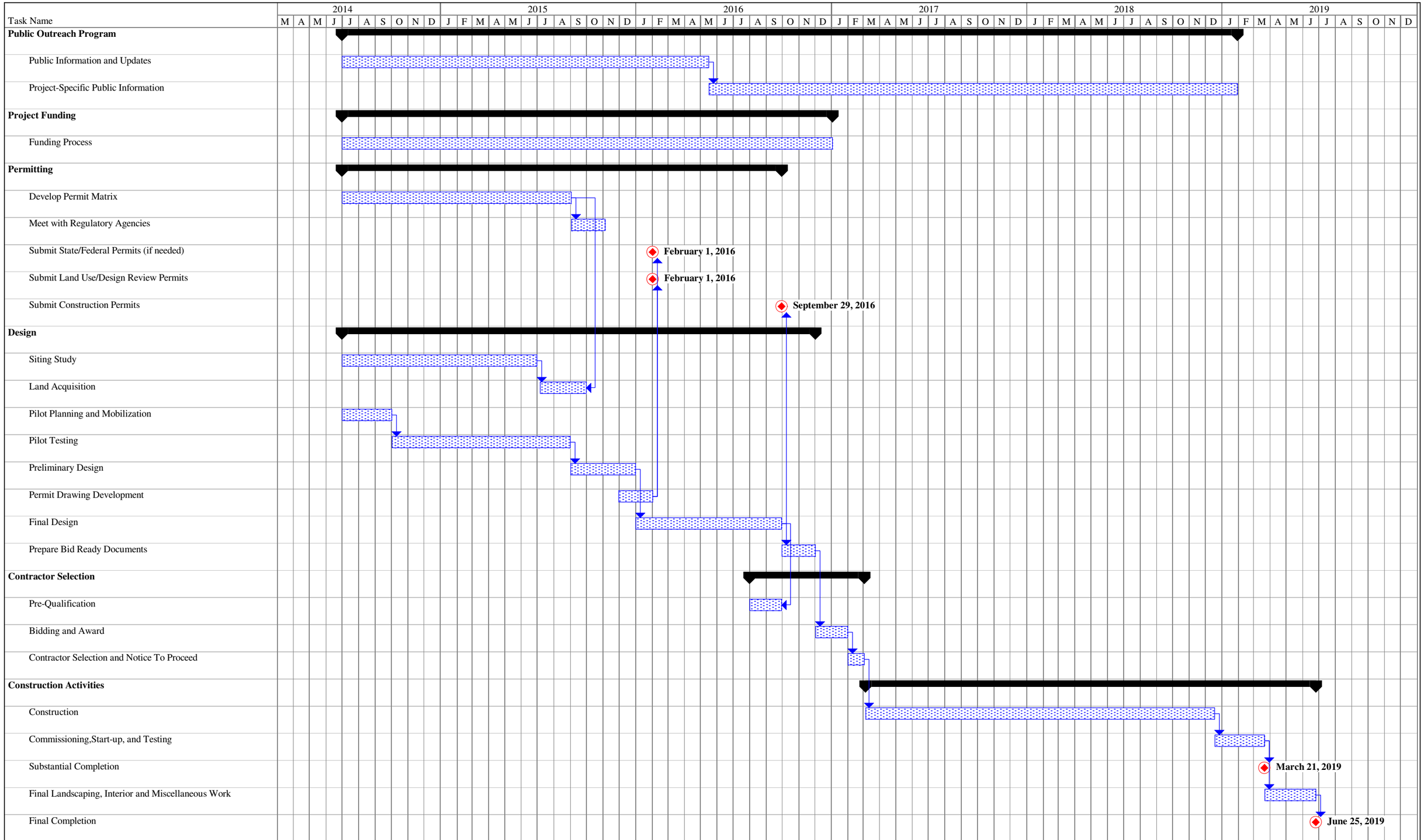


Figure 9-3: Grants Pass WTP Conventional DBB Implementation Schedule - DRAFT

Table 9-11
Recommended Capital Improvement Program Summary[1,2,3,4]

CIP Year	1	2	3	4	5	6	7	8	9	10	Anticipated CIP Expenditures for Project Component
Fiscal Year	**2014-15**	**2015-16**	**2016-17**	**2017-18**	**2018-19**	**2019-20**	**2020-21**	**2021-22**	**2022-23**	**2023-24**	
New Water Treatment Plant Implementation											
Pilot Plant Study	$400,000	$100,000									$500,000
Siting Study and Property Acquisition	$200,000	$1,100,000									$1,300,000
Funding Study and Rate Impact Study[5]	$100,000	$100,000									$200,000
Project Implementation Approach and Procurement Strategy	$50,000										$50,000
Public Information/Involvement	$50,000	$50,000	$50,000	$50,000	$50,000						$250,000
Permitting and Land-Use Approvals	$50,000	$75,000	$75,000								$200,000
Preliminary Design		$1,000,000									$1,000,000
Final Design		$1,000,000	$3,000,000								$4,000,000
Bidding and Award			$250,000								$250,000
Construction			$10,200,000	$18,500,000	$18,500,000						$47,200,000
Post-Construction and Warranty Period						$100,000	$100,000				$200,000
Existing Water Treatment Plant Investments											
Emergency Response Plan	$50,000										$50,000
Decommission and Demolition of Existing Plant[6]							$250,000	$250,000	$250,000	$250,000	$1,000,000
Total Anticipated Annual Expenditures	**$1,000,000**	**$3,525,000**	**$13,375,000**	**$18,550,000**	**$18,550,000**	**$100,000**	**$350,000**	**$250,000**	**$250,000**	**$250,000**	**$56,200,000**

Notes
1. Schedule assumes design-bid-build project delivery.
2. From fiscal year 2014-15 to 2023-24, based on Alternative 4 project costs
3. All costs are in 2013 U.S. dollars.
4. This is a Class 5 estimate. The accuracy ranges from –30 percent to +50 percent.
5. Funding and rate impact study costs assume that separate studies are not performed for the distribution system capital improvements program.
6. Costs for decommissioning and demolishing the existing water treatment plant were not included in the project costs presented in Chapter 8.

Project Initiation Activities

It is recommended that the City accomplish the following tasks during the first year of planning for construction of a new WTP:

- Develop a funding strategy
- Select a site for the new WTP
- Conduct a pilot plant study to evaluate high-rate filtration, high-rate clarification processes, and intermediate ozonation processes
- Confirm the project schedule and project delivery strategy
- Plan and implement a public outreach program
- Develop a permitting and regulatory approval plan

It is anticipated that the City will need to allocate approximately $1 million to complete these activities. Once significant progress has been made on each of these tasks, the detailed design phase may begin. Each of the tasks is briefly described below.

Funding Strategy

The City will need to decide how to fund this large capital improvement project. Impacts to customer rates from the WTP project will need to be determined. The rate study should also consider the financial impacts of other potential water system capital improvement projects which will be determined during preparation of the upcoming Water System Master Plan. This effort should begin as soon as possible and will take at least 12 months to complete, depending on when the Water System Master Plan CIP is finalized.

Site Selection

Per Council direction, the City needs to evaluate potential locations for the new WTP and then select a preferred site for acquisition. This task should be initiated as soon as possible and will likely take 12 to 18 months to complete.

There are currently a number of potential sites near the existing WTP which are considered suitable. This includes the property across the street from the existing WTP. This property is currently owned by the City.

After an initial screening of potential sites, testing should be performed at the selected properties to assess geotechnical conditions, determine whether hazardous materials are present, and identify anything else which may present obstacles to developing the property. The siting study should include a permitting review to identify potential permitting issues and development conditions.

Pilot Plant Study

In order to take advantage of the lower capital costs and smaller space requirements offered by high-rate clarification and filtration processes, a pilot plant study is needed to proof-test these processes with Rogue River water. The OHA requires a one-year long pilot plant study for use of filtration rates above 6 gpm/sf. Continuous pilot testing of alternative clarification technologies, such as ballasted flocculation, throughout the year may not be required, but a "reasonable" duration of testing during each season is necessary. This duration can range from 4 to 8 weeks per season depending on a number of testing and performance evaluation parameters. It is also recommended to pilot test the use of intermediate ozonation to determine its impacts to water quality and other processes.

Pilot testing should ideally be conducted on a seasonal basis to determine performance under variable water quality conditions which are experienced at the existing WTP, especially winter, summer and fall/transitional periods. If the City begins the pilot testing work in July 2014, the testing can be completed by spring 2015 and the final reporting completed during summer 2015.

The following tasks are suggested as part of the pilot plant study:

- Develop a testing plan, determine equipment needs, and confirm budget
- Procure equipment, deliver to site and install
- Seasonal pilot testing and data collection
- Reporting, including interim reports after first two seasons of testing
- Report submittal to OHA and review meeting
- Confirmation of treatment process selection

The costs to complete a pilot plant study can be highly variable and depend on factors like equipment costs and labor assigned to operate and monitor the pilot plant equipment. The most economic approach is to have City staff assume the daily operations and data collection duties after receiving training and startup assistance from consultant staff and equipment suppliers.

Project Schedule and Delivery Method

The City needs to confirm the appropriate project schedule for the new WTP and to verify the desired method to deliver the project. The schedule will ultimately depend on the method of project delivery. There are multiple project delivery options for the City to consider in addition to a traditional design-bid-build approach. For example, design-build may allow earlier completion. The City may also consider a public-private partnership. Public-private partnerships are becoming more common for large capital projects because they are partially or completely funded by a private party as part of the program.

In addition to project schedule and method of project delivery, the City should consider the following:

- Whether improvements to the water supply system which are necessary to integrate the new WTP should be designed and constructed as unique projects or completed as part of the WTP project
- Early procurement of key process equipment
- Long-term strategy for operating the new WTP (continuing with City staff operation or using a third party as in design-build-operate)

Public Outreach

Public support will be an important component in the overall success of the project. Experiences from recent similar projects in the region has shown that the public is interested in and aware of its source of drinking water supply and will be very active in expressing their opinions in this matter.

The City has laid the foundation for a very open, transparent, and active public information program over the past year to keep the public updated on the various water supply alternatives. This program should continue until long after the new plant is constructed to ensure a level of transparency that the community demands. As the City has gathered all of the information needed to properly evaluate all of its future water supply alternatives, the information program should be expanded to include a range of activities from a broader public education campaign to inclusion of a public involvement component to assist in the final design decision-making process as it relates to public amenities.

Permitting and Regulatory Approvals

The City should develop an inventory of permitting requirements and submittals that will be required for the project. Assignments of responsibility should be made to ensure that all of the required permits and regulatory approvals are obtained within the appropriate time frame. The scope of permitting will become clearer as the level of project definition increases. This task is of critical importance and should begin in the first year of planning. Experience has shown that permitting can take longer than any other part of a project. Failure to address permitting issues early enough in the project can delay the schedule.

Preliminary Design Activities

Once a site has been acquired, the funding method has been determined, and pilot testing has been completed, the next steps in the plant design process will include:

- Development of Basis of Design Report (BoDR) which reflects approximately 10 percent design completion
- Initial opinion of probable construction cost (OPCC) based on BoDR

- Continuance of public outreach program
- Refinement of permitting requirements

It is anticipated that this activity will begin during fiscal year 2 in fall 2015 and will take approximately four months to complete.

Final Design Activities

After preliminary design activities have been completed, final design will commence. Final design tasks include:

- Early equipment procurement, if determined to be beneficial to the project
- Development of detailed plans, specifications, and bidding documents
- Additional OPCCs at selected intermediate and final design stages
- Project permitting and approvals
- Continuance of public outreach program

It is anticipated that this activity will begin during the latter half of fiscal year 2 in early 2016 and will take approximately 10 months to complete. This would allow bidding to begin in fall 2016 during fiscal year 3.

Bidding and Award

After final design has been completed, bidding activities will commence. These activities will include:

- Pre-qualification of bidders
- Advertisement for bids and pre-bid meeting with prospective bidders
- Receipt of questions from prospective bidders and issuance of addenda as necessary
- Receipt and review of bids
- Recommendation of award to apparent low bidder
- Project permitting and approvals, if needed
- Continuance of public outreach program

With bidding set to begin in fiscal year 3, it is anticipated that this activity will take approximately three months to complete to allow the award to be made in the latter half of fiscal year 3. This schedule and list of tasks assumes that a design-bid-build project delivery will be used. Conditions could change if the City uses a different project delivery method.

Construction, Startup, and Commissioning

A 28-month construction duration is anticipated which would provide for final completion in June 2019. The last few months of this activity include time for startup, testing, commissioning, and operator training. It is anticipated that the City will desire a two-year

warranty period which would conclude in 2021. The project costs for this activity include construction, inspection, construction management, and engineering services during construction.

Investments in the Existing Water Treatment Plant

Since a new WTP will be constructed, the City should limit its investment in the existing WTP. With the exception of the intake structure, investments in the existing WTP structures would be lost as soon as the new WTP is online and the existing WTP is decommissioned. However, the City should budget some money for the existing WTP including the following items:

- The existing intake may require modifications to improve the handling of silts and solids which accumulate in the pumping wetwell. The City has completed designs for low-cost upgrades of the de-silt system to help alleviate these issues. Completion of these improvements or a more permanent, long-term improvement should be deferred and re-evaluated based on observation of siltation in the spring of 2014 and the overall sequencing of the Project Initiation Activities described earlier in this chapter.
- The City will need to determine the ultimate fate of the existing WTP after the new WTP becomes operational. A budget of $1 million has been allocated for decommissioning and demolition activities in the final four years of the CIP presented in Table 9-11.
- The City should develop an Emergency Response Plan. Specific attention should be given to areas of the plant which are highly susceptible to partial or complete failure in a seismic event.

Summary

The City of Grants Pass should immediately begin the process to construct a new WTP due to the age and structural condition of the existing WTP. In order to minimize the risks to the City's only drinking water supply, and to reduce continued investment in the existing plant, the City should plan to have a new WTP online in 2019. The estimated project cost to plan, design, and build a new WTP is $56.2 million. This project cost will be incurred in capital expenditures made over the next 10 years.

Critical early planning activities should begin in the next fiscal year to ensure that the new WTP is online in 2019. The City should budget approximately $1.0 million for this initial planning work which includes site selection, a pilot plant study, and a funding analysis.

The City will need to determine how to pay for this significant investment and should also consider potential investments in its distribution system, which will not be identified until after the upcoming Water System Master Plan is completed. A public outreach program can help the City engage its citizens to help explain why these investments are important to the community.

APPENDIX

APPENDIX A

PUBLIC HEALTH DIVISION
Drinking Water Program
John A. Kitzhaber, MD, Governor

Oregon Health Authority

2860 State Street
Medford, OR 97504
(541) 618-7872
FAX: (541) 776-6013
scott.g.curry@state.or.us

December 05, 2012

Michael McWhirter
MWH
806 SW Broadway, #200
Portland, OR 97205

RE: Plan Review #187-2012 / Approval
Disinfection Tracer Study / City of Grants Pass (PWS #4100342)

Dear Mr. McWhirter:

We have received a copy of the Disinfection Contact Time Tracer Study that was conducted at the Grants Pass water treatment plant between July and September 2012.

We note that the protocol for this study was submitted and approved earlier, and the objective of the study was to evaluate the contact time available in the sedimentation basins at low, medium, and high (95% of peak) flowrates. Contact time in the clearwell has been determined with a previous study.

We agree with the results of this tracer study, and it is approved. The City may start using the new contact time (T10) numbers for reporting purposes in the monthly surface water treatment report. Should the plant flowrate exceed that of the study, a new tracer study will be required.

Thank you for submitting this report for our review. We appreciate the City's efforts to ensure their compliance with surface water treatment regulations.

Sincerely,

Scott G. Curry, P.E.
Regional Engineer
Drinking Water Program

cc: Jason Canady, City of Grants Pass

MEMORANDUM

TO: Jason Canady, City of Grants Pass

DATE: November 7, 2012

FROM: Michael McWhirter, MWH
Andrew Nishihara, MWH

CC: Brian Ginter, MSA
Chris Kelsey, MSA

REVIEWED BY: Jude Grounds, MWH
Peter Kreft, MWH

SUBJECT: Grants Pass WTP – Tracer Test Results

Executive Summary

Tracer tests for the sedimentation basins were conducted at the City of Grants Pass Water Treatment Plant (Grants Pass WTP) from July to September 2012. These tests were requested by the Oregon Health Authority (OHA) following the Water System Survey and WTP Inspection conducted in June 2011. Historical calculations of chlorine disinfection performance at the Grants Pass WTP were partially based on estimates of hydraulic efficiency through the sedimentation basins without supporting tracer test data. This technical memorandum summarizes the results of these tests.

The hydraulic efficiency values obtained through tracer testing ranged from 0.46 to 0.57 for the three flowrates tested. These results will be used for future disinfection performance (CT) calculations by plant staff, and produce similar results to the historical CT calculation method. A CT model was created for use by the plant staff based on these findings; operators can input plant flow rate, pH, chlorine residual and water temperature and determine the resulting inactivation of *Giardia*. Staff can use this model to proactively manage operation of the WTP to ensure continued compliance with disinfection requirements.

Introduction

A series of tracer tests were performed at the WTP by plant staff between July and September 2012. MWH assisted in the preparation of the tracer test plan and also in analysis and verification of the results. The tracer tests were performed to meet the requirements of the Surface Water Treatment Rule (SWTR), which was promulgated by the United States Environmental Protection Agency (EPA) in 1989. The SWTR was then augmented by the Interim Enhanced Surface Water Treatment Rule in 1999 and the Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) in 2006. The objective of these regulations is to protect the public from exposure to waterborne pathogens, particularly *Cryptosporidium* oocysts, *Giardia* cysts, *Legionella* and viruses, which can be found in surface water supplies. The regulations require all utilities served by a surface water supply to achieve a minimum 3-log reduction of *Giardia* cysts, a 4-log reduction of viruses and a 2-log reduction of *Cryptosporidium* oocysts during the treatment of drinking water. The reductions in *Giardia*, bacteria and viruses are to be achieved through a multi-barrier approach of both physical removal and chemical inactivation. Under the LT2ESWTR, utilities were required to provide additional reduction of *Cryptosporidium* if their raw water source was found to be susceptible to elevated concentrations of those protozoa.

The Grants Pass WTP is classified and operated as a conventional filtration plant. The OHA has awarded the Grants Pass WTP with a 2.5-log removal credit for *Giardia*, a 2.0-log removal credit for viruses and a 2.0-log removal credit for *Cryptosporidium* at all plant flows up to its maximum production rate of approximately 20 mgd. The Grants Pass WTP is required to achieve the remaining level of *Giardia* and virus reduction via disinfection. Thus, disinfection at the plant must achieve a minimum of 0.5-log inactivation of *Giardia* and 2.0-log inactivation of viruses for compliance with the existing regulations. Sampling of the *Cryptosporidium* concentrations in the Rogue River, which is the source for the Grants Pass WTP, has shown that no additional removal/disinfection of *Cryptosporidium* is required at this time.

In order to determine the level of inactivation achieved during chemical disinfection, the EPA developed the "CT" concept. "CT" is the product of the disinfectant residual concentration (C) measured at the outlet of a disinfection section(s) and the time (T) representing the minimum detention time experienced by 90% of the water passing (or the time in which 10 percent of an added tracer passes through the section(s)). This time period is commonly referred to as the T_{10}. Tables are provided in the SWTR Guidance Manual identifying the minimum level of CT required to achieve various levels of *Giardia* and virus inactivation based on site specific water quality conditions. The EPA recommends that utilities conduct a tracer test to determine the value of T_{10} available in the disinfection section(s) which will be used for CT compliance.

As a result of a Water System Survey and WTP Inspection conducted in June 2011, the OHA requested that the City complete tracer testing at the WTP to demonstrate the hydraulic efficiency which is achieved through the plant. The City has previously conducted tracer tests for the clearwell; consequently the tests conducted for this report investigated the portion of the WTP between the static mixer, where sodium hypochlorite is first introduced to the water, and the Combined Filter Effluent (CFE) point upstream of the clearwell.

The tracer tests were conducted at three flowrates: 9.4 mgd, 15.1 mgd, and 19 mgd. The flowrates represent low, average, and high (95% of peak flowrate), operating rates at the plant.

WTP Process Description

The Grants Pass WTP is rated a conventional filtration water treatment plant with a maximum capacity of approximately 20 mgd. Raw water is pumped from the adjacent intake on the Rogue River into three sedimentation basins. Valve positions are set to split the flow between the basins based on basin size. Coagulation at the plant is achieved by dosing alum in conjunction with aluminum chlorohydrate (ACH) or poly-aluminum chloride (PACl). Filter aid polymer is added to the settled water to improve filter performance. Disinfection is achieved through pre- and post-filtration chlorination using 12.5% sodium hypochlorite. Potassium permanganate is used occasionally to control taste and odor in the finished water. The plant is frequently operated for less than 24 hours a day when there is low demand for water.

Tracer Test Methodology

Per Oregon Administrative Rules (OAR), the City was required to submit a tracer test plan to the State for approval prior to conducting any tracer tests. The testing plan was completed and submitted in March 2012. Tracer tests were conducted by injecting a solution of calcium chloride ($CaCl_2$) for the 'slug dose' method. Testing under the three flow conditions (low, average and high) as prescribed by the testing plan was completed in July 2012. Additional testing for verification and to confirm results continued intermittently through September 2012.

Tracer testing was conducted in all three of the plant's sedimentation basins at the same time. The slug dose of $CaCl_2$ was added to the flash mixer immediately downstream of coagulant addition and just prior to sodium hypochlorite addition.

Preliminary testing showed that a slug dose of 200 lbs of $CaCl_2$ dissolved with 40 gallons of water was adequate to increase the conductivity of the water high enough above the baseline conductivity to allow accurate analyses. The tracer solution was made up in the plant's polymer batching tanks. Due to the exothermic reaction from dissolution of $CaCl_2$ in water, an ice bath was used to cool the tracer solution before being used in the tests.

Successful slug dose testing introduces the full volume of dosing solution to the water in less than two percent of the hydraulic residence time (HRT) of the portion of the plant being tested. Based on the maximum test flow rate of 19 mgd, this required the tracer solution to be added in less than 2.5 minutes. The more quickly the solution is introduced, the more accurate and reliable the results. The injection system used by the plant staff was able to introduce the tracer solution in approximately 10 seconds or less.

The City of Grants Pass used their HACH conductivity monitor, with six separate sensors, to gather data points at one-minute intervals during the tracer test. Conductivity was measured at the following locations:

1. the inlet to the mixing basin which feeds Sedimentation Basins 1 and 2,
2. the inlet to Sedimentation Basin 3
3. the effluent from Sedimentation Basin 1,
4. the effluent from Sedimentation Basin 2,
5. the effluent from Sedimentation Basin 3, and
6. the CFE immediately upstream of the clearwell.

The sample probes for locations 1-5 were immersed directly into the basins. Water for sample 6 was fed to the conductivity probe through a sample pump and a short length of sample tubing.

The background conductivity was measured at all six conductivity sample points prior to the test, for a period of thirty minutes, to ensure that the background conductivity was stable. The tests were conducted during a period of stable raw water quality and flowrate to ensure there was no need to change the dosing rates of any of the treatment chemicals during the test. Approximately once per hour, grab samples were collected from each of the six locations for conductivity spot checks using the WTP's laboratory conductivity analyzer to verify the on-line conductivity measurements.

All three sedimentation basins and all eight filters were in service throughout each of the tests. No backwashing or recycling occurred during the tests. Each tracer test lasted for at least two HRTs and was conducted until sampling and calculations showed that close to 100% of the tracer had been passed.

Table 1 shows approximate HRTs through the various parts of the plant based on dimensions and volumes of basins, filters, and their related piping.

Table 1: Approximate Hydraulic Residence Times

	Hydraulic Residence Time (minutes)		
Segment of Plant	**@ 9.4 mgd**	**@ 15.1 mgd**	**@ 19 mgd**
Sedimentation Basins	247	153	122
Filters[1]	12	8	6
Total	259	161	128

[1] This estimate of the hydraulic residence times through the filters is based on a porosity in the filter media of 50% and minimum assumed water level above the top of the media of 4-feet .

Due to the long period of time necessary to obtain results from the test at the minimum flow rate, the 19 mgd test was conducted first. The 19 mgd test lasted for 287 minutes (2.2 HRT), the 15.1 mgd test lasted for 422 minutes (2.6 HRT), and the 9.4 mgd test lasted for 963 minutes (3.7 HRT).

Results

Appendix A presents plots of conductivity versus time for the WTP CFE for the tests. Time is plotted on the x-axis where '0' represents the time when the tracer solution was introduced. The primary y-axis on the left side of the plots indicates the percent of tracer solution which had passed the CFE measuring location, while the secondary y-axis on the right side of the plots shows the instantaneous conductivity measurement of each sample point. A single point, highlighting the time when 10% of the tracer solution had passed (T_{10}), was placed on each of the plots based on the observation that 100% of the tracer was recovered once the conductivity drops to background levels. $T_{theoretical}$ (T_{th}) values were equivalent to the HRT for each flowrate. As shown in the plots, the measured peak conductivity after slug addition was at least 38% higher than the measured background conductivity for each of the tests. The calculated T_{10}/T_{th} (hydraulic efficiency) values based on the test results are summarized in **Table 2** as follows:

Table 2: Tracer Test Results

Date	Flow Rate (mgd)	T_{10}/T_{th} (CFE)
7/5/12	19.0	0.48
7/6/12	15.1	0.46
7/18/12	9.4	0.57

The hydraulic efficiencies shown for the CFE are similar under the average and high flow conditions. At the low flow condition, the hydraulic efficiency increased. The measured values are consistent with values that have been measured at similar WTPs in the region. The most conservative approach for the WTP would be to use the lowest hydraulic efficiency measured for all flowrates. However, the rules allow for interpolation between data points based on flow, as is currently practiced for the Clearwell CT calculations. For consistency in CT calculation methodology, we recommend that the plant use the values from this study for the pre-Clearwell CT calculations as summarized in **Appendix B: Table B-1**.

Conclusions

Using the tracer test results for the different flow rates and the EPA's "*Compliance with the Filtration and Disinfection Requirements for Public Water Systems Using Surface Water Sources Guidance Manual*", a profile was developed to interpolate and extrapolate additional T_{10} values to be used for CT calculations at the plant for the 10 currently approved operating flows. The graph and table of these values can be found in **Appendix B**.

As discussed previously, the plant must achieve a minimum of 0.5-log inactivation of *Giardia* and 2.0-log inactivation of viruses for compliance. When using free chlorine as the primary disinfectant, *Giardia* inactivation requirements are higher than the virus inactivation requirements under all conditions.

Water temperature fluctuations have the biggest impact on *Giardia* inactivation with free chlorine, followed by pH and then chlorine residual. The lowest recorded raw water temperature since 2004 has been 0.7°C, and there is a potential for the plant to experience 0.5°C water. The raw water temperature is typically less than 5°C approximately 18 days per year.

Table 3 presents a summary of maximum plant flows to meet CT over a range of water temperatures, as calculated by a CT model. The 0.7 log column represents the WTP's internal disinfection benchmark, while the 1.0 log column shows an additional conservative condition. This analysis assumed the following conservative water quality parameters based on historical plant operating data:

- Sedimentation Basin pH = 7.5
- Sedimentation Basin effluent chlorine residual = 0.1 mg/L
- Clearwell pH = 7.5
- Clearwell effluent chlorine residual = 0.9 mg/L

Table 3: Maximum Plant Flow Rates for Various Temperatures

Temp (°C)	Max Flow that achieves CT = 0.5 log DS of Filters (mgd)	Max Flow that Achieves CT = 0.5 log Total (mgd)	Max Flow that Achieves CT = 0.7 log Total (mgd)	Max Flow that Achieves CT = 1.0 log Total (mgd)
0.5	5.6	8.3	6.0	4.1
5	8.0	11.7	8.4	5.9
10	10.6	15.7	11.2	7.9
15	15.5	20+	16.4	11.4
25	20+	20+	20+	20+

DS – downstream
mgd – million gallons per day

Figure 1 visually displays the modeled log inactivation of *Giardia* through the WTP and clearwell over a range of water temperatures.

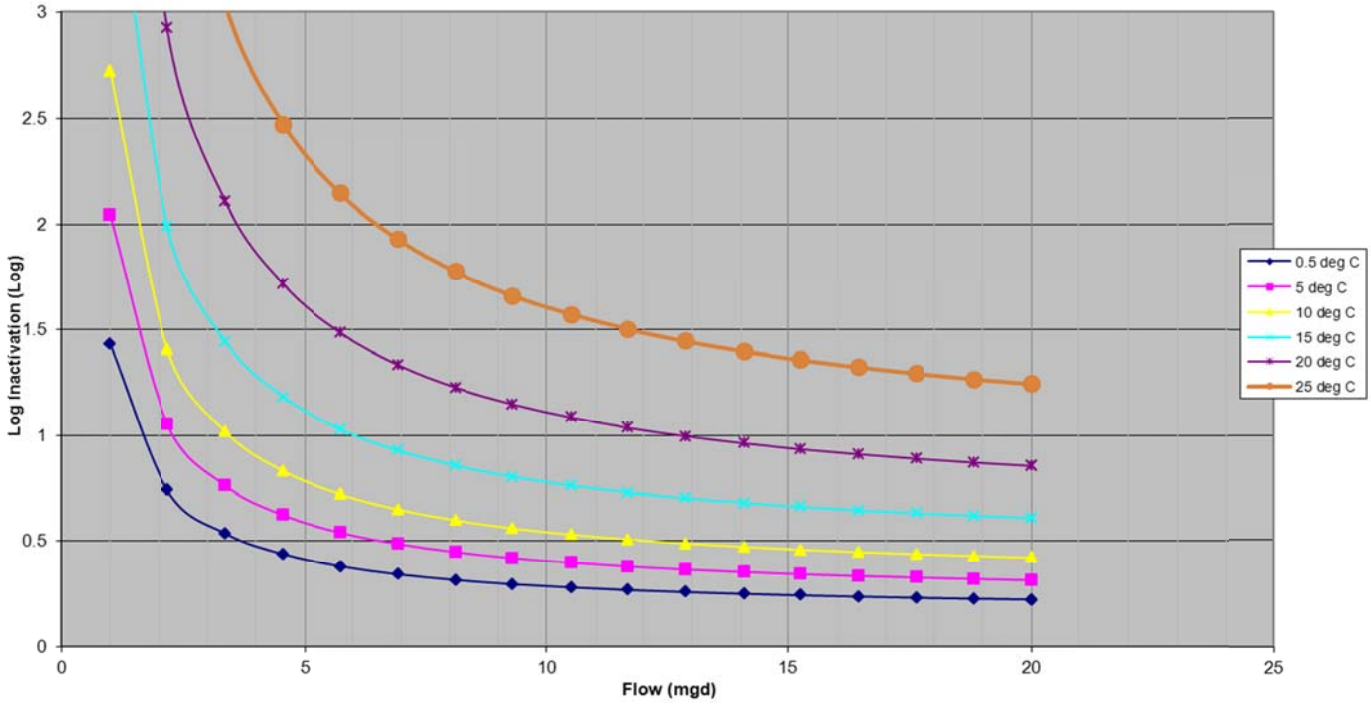


Figure 1: Log inactivation of *Giardia* through the WTP and Clearwell

Under these conservative conditions the calculations demonstrate that the WTP can continue to meet the required 0.5-log inactivation of *Giardia* while meeting customer demands. If/when winter demands increase, the WTP will need to adjust its disinfection practices by increasing the chlorine residual, or expanding the Clearwell volume, or installing an alternative disinfectant like UV irradiation.

[NOTE: A CT model was developed for the Grants Pass WTP, and was used to determine Giardia inactivation (since Giardia disinfection requirements are more onerous than virus for Grants Pass WTP) under different water quality and operating conditions. This model incorporates CT values for free chlorine as presented in the EPA's "Surface Water Treatment Rule Guidance Manual". MWH has successfully used this model for other plants throughout the nation to optimize CT performance. A copy of the model and the User's Guide is included in ***Appendix C****. Though this CT Model will not replace OHA-prescribed CT calculations at the plant, in the future, the model may be used by WTP staff to optimize chlorine use at the plant.]*

Appendix A: Tracer Test Results

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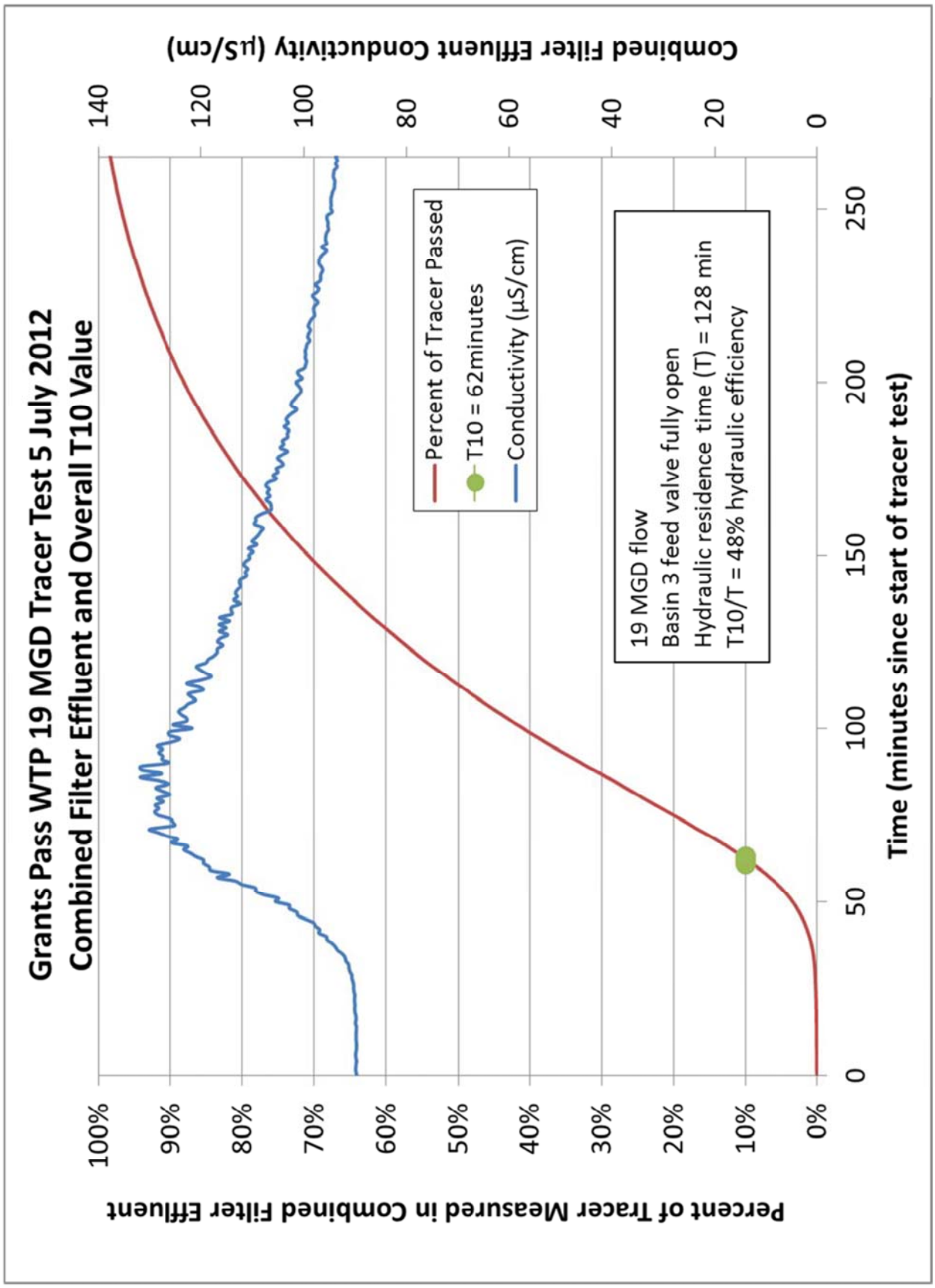
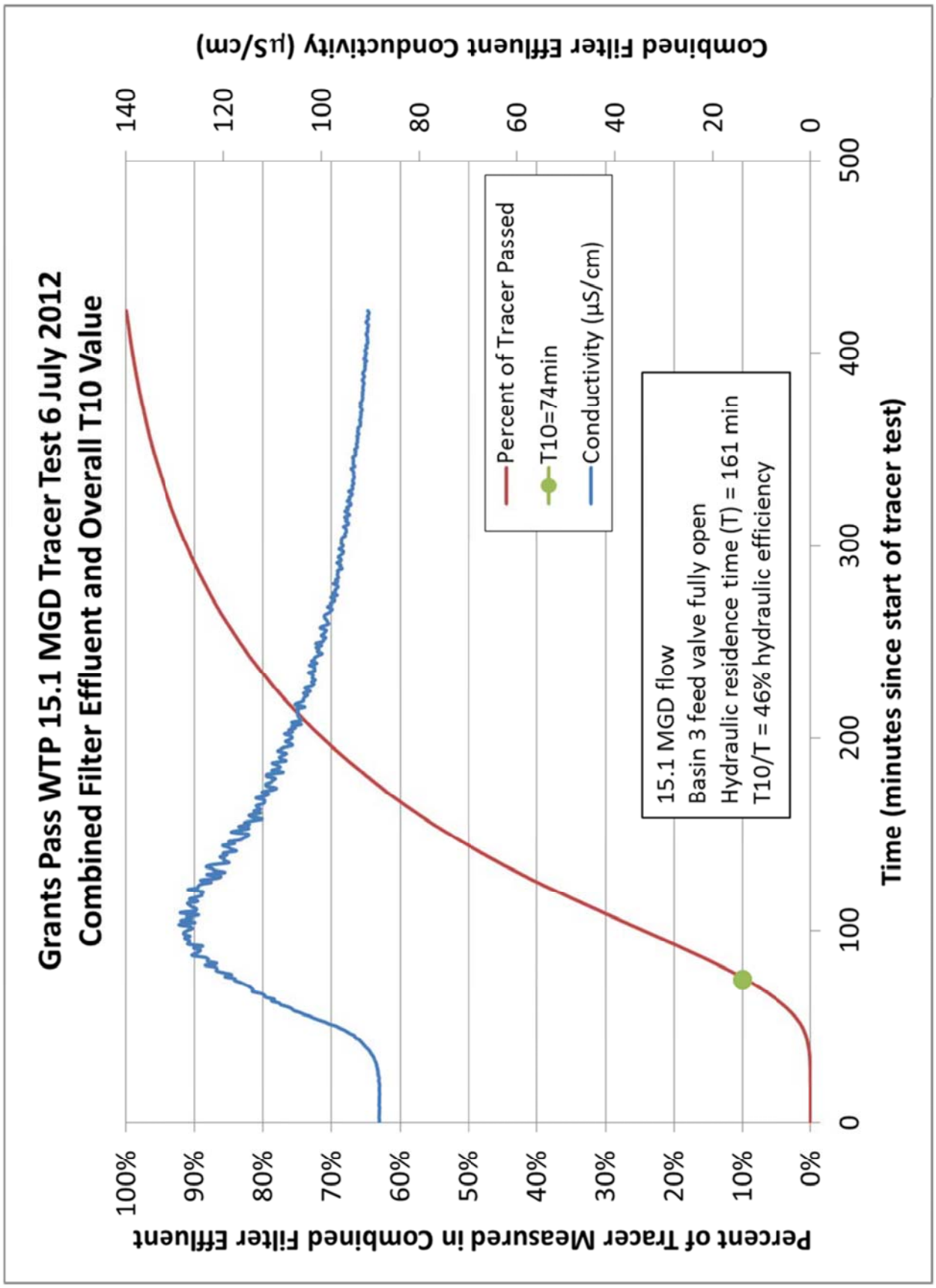
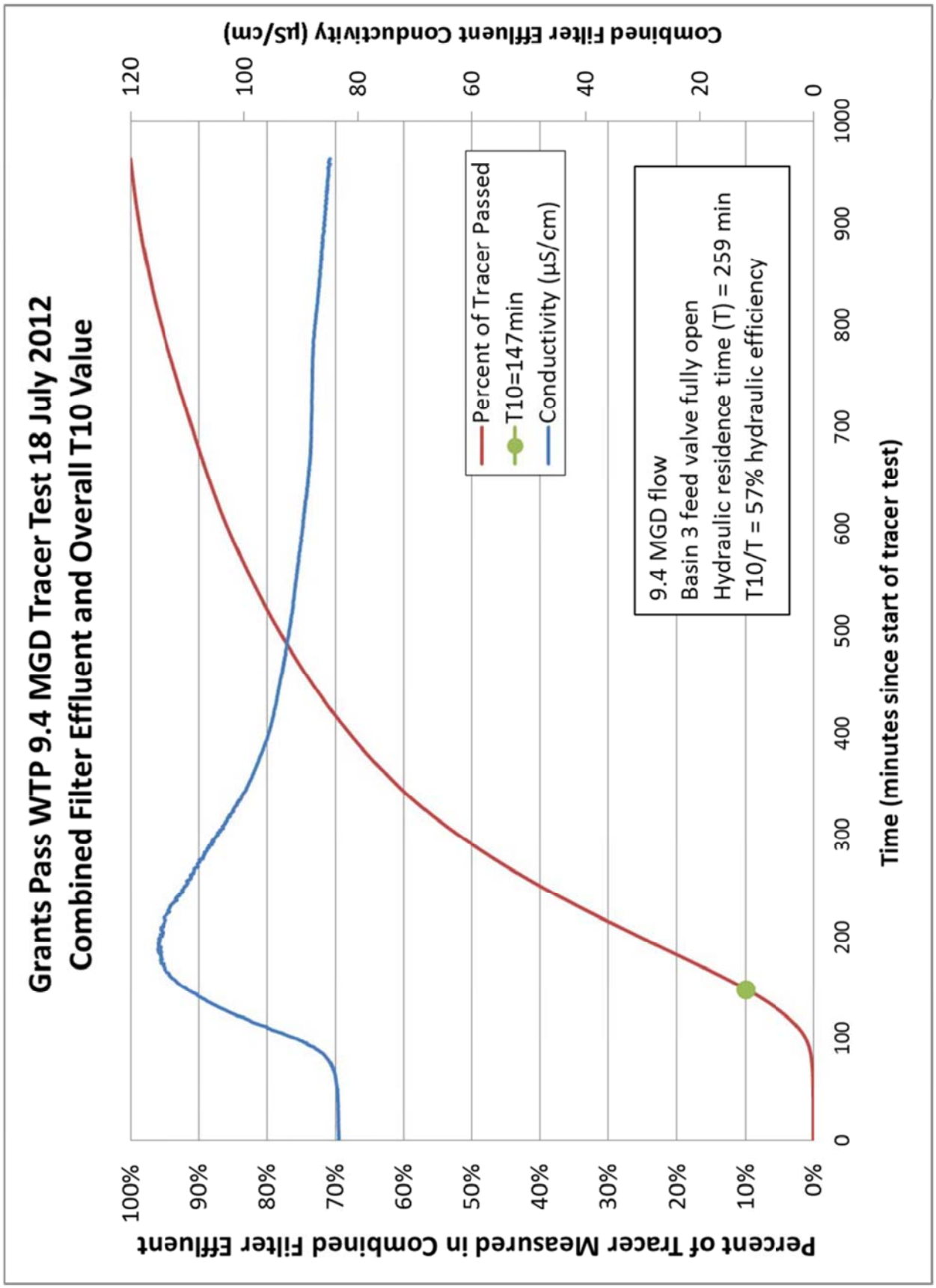

Grants Pass WTP 19 MGD Tracer Test 5 July 2012
Combined Filter Effluent and Overall T10 Value
Percent of Tracer Passed
T10 = 62minutes
Conductivity (µS/cm)
19 MGD flow
Basin 3 feed valve fully open
Hydraulic residence time (T) = 128 min
T10/T = 48% hydraulic efficiency
Percent of Tracer Measured in Combined Filter Effluent
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
Combined Filter Effluent Conductivity (µS/cm)
140
120
100
80
60
40
20
0
Time (minutes since start of tracer test)
0
50
100
150
200
250


Grants Pass WTP 15.1 MGD Tracer Test 6 July 2012
Combined Filter Effluent and Overall T10 Value
Percent of Tracer Passed
T10=74min
Conductivity (µS/cm)
15.1 MGD flow
Basin 3 feed valve fully open
Hydraulic residence time (T) = 161 min
T10/T = 46% hydraulic efficiency
Percent of Tracer Measured in Combined Filter Effluent
Combined Filter Effluent Conductivity (µS/cm)
Time (minutes since start of tracer test)
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
140
120
100
80
60
40
20
0
0
100
200
300
400
500


Grants Pass WTP 9.4 MGD Tracer Test 18 July 2012
Combined Filter Effluent and Overall T10 Value
Percent of Tracer Passed
T10=147min
Conductivity (µS/cm)
9.4 MGD flow
Basin 3 feed valve fully open
Hydraulic residence time (T) = 259 min
T10/T = 57% hydraulic efficiency
Percent of Tracer Measured in Combined Filter Effluent
Combined Filter Effluent Conductivity (µS/cm)
Time (minutes since start of tracer test)
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
120
100
80
60
40
20
0
0
100
200
300
400
500
600
700
800
900
1000

Appendix B: Hydraulic Efficiency Values

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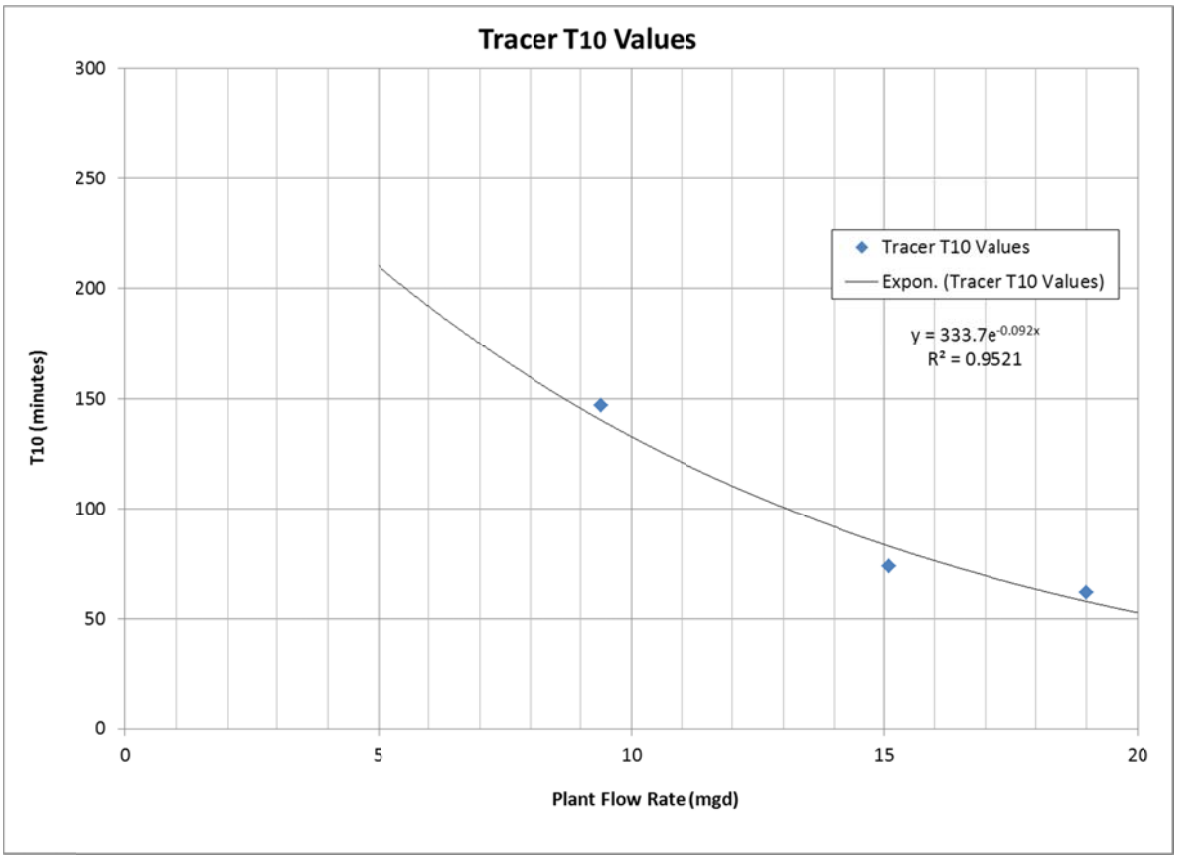

Figure B-1: Plot of 3-value T_{10} Curve

Table B-1: Tracer T_{10} and T_{10}/T_{TH} Values for CT Calculations

Flowrate (mgd)	T10 (mins)	T_{10}/ T_{TH}
5.1	208	0.44
6.5	184	0.49
7.9	161	0.53
9.4	141	0.54
10.5	127	0.55
12.2	108	0.55
13.7	95	0.53
15.1	83	0.51
16.6	72	0.48
18.0	63	0.47
19.0	58	0.45
19.4	56	0.44
20.0	53	0.44

Appendix C: CT Model User Guide

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MEMORANDUM

TO: Jason Canady, City of Grants Pass **DATE:** November 7, 2012

FROM: Michael McWhirter, MWH **CC:** Brian Ginter, MSA

Andrew Nishihara, MWH Chris Kelsey, MSA

REVIEWED BY: Jude Grounds, MWH

SUBJECT: CT Model – User Guide – Draft

Introduction

The City of Grants Pass Water Treatment Plant (WTP) CT Model is intended for two purposes: 1) to serve as a tool for doing "what if" analyses when optimizing the plant's disinfection strategy, and 2) to assist plant operators in accurately calculating and consistently reporting daily CT compliance. The model is interactive, and should be modified to best represent the treatment conditions of interest. The model breaks the plant into several "disinfection sections", and uses look-up tables to calculate the log-removal of *Giardia lamblia* through that particular section. Overall disinfection performance is then calculated by summing the disinfection achieved through the individual sections of the plant.

This Users Guide presents regulatory and theoretical background necessary for determining disinfection compliance, as well as a step-by-step guide to operating the model and interpreting the results.

Background

The following section provides background information on the disinfection requirements for the Grants Pass WTP and a review of the calculations involved when reporting inactivation performance.

CT Requirements

Drinking water requirements, as defined under the Surface Water Treatment Rule (SWTR), are defined in Oregon law under OAR 333-061. The objective of OAR 333-061 is to protect the public from exposure to pathogenic organisms, particularly *Giardia lamblia* cysts, *Legionella*, and viruses, which can be found in surface water supplies. By law, all utilities served by a surface water supply are required to achieve a minimum of 99.9% (3-log) reduction in *Giardia lamblia* cysts and 99.99% (4-log) reduction in viruses during drinking water treatment. Removal credit is awarded to water treatment plants based on the types of processes provided. For plants rated conventional such as the Grants Pass WTP, a 2.5-log and 2.0-log removal credit is achieved for *Giardia lamblia* and viruses, respectively. The remaining reduction in pathogenic

organisms must come in the form of disinfection. For the Grants Pass WTP, a minimum of 0.5-log inactivation of *Giardia*, and 2-log inactivation of viruses is required prior to the first customer.

To determine the level of inactivation achieved during chemical disinfection, the EPA developed the "CT" concept. Compliance with the disinfection requirements is achieved when the following equation is true:

1. $\mathbf{CT_{achieved} \geq CT_{required}}$

Where: $CT_{achieved}$ through the treatment process is calculated by the product of the disinfectant residual concentration measured at the outlet of a disinfection section ("C") and the "effective" detention time through a section of the plant ("T"), commonly referred to as the T_{10}. Additional discussion of the T_{10} is presented in a later section.

$CT_{required}$ can be determined using tables provided in the SWTR Guidance Manual, which identify the minimum level of CT that is required to achieve various levels of *Giardia lamblia* and virus inactivation. This value is a function of the following water quality parameters, as measured at the end of the disinfection section:

- Disinfectant residual
- pH
- Temperature

The Grants Pass WTP relies on free chlorine contact time through the plant to achieve the necessary CT.

Calculating T_{10}

The theoretical detention time (T_{th}) through a basin (in this case, a disinfection section) represents the average time a molecule of water spends in that basin, and is calculated using the following equation:

2. $$\mathbf{T_{th}\ (min) = \frac{Disinfection\ Section\ Volume\ (gal)}{Flow\ Rate\ (gal/min)}}$$

T_{10}, or the effective detention time is used when calculating "CT" to better account for any "short-circuiting" that may occur through a particular disinfection section, and is typically a fraction of the theoretical detention time. T_{10} is defined as the time in which 10% (by volume) of an added tracer would passes through the outlet of the disinfection section. Thus, the ratio of T_{10}/T_{th}, or hydraulic efficiency (*e*) is always less than or equal to 1, depending on the mixing characteristics in the disinfection section. For example, well mixed portions of the treatment process (rapid mix, for example) will have a hydraulic efficiency of approximately 0.1—only 10% of the theoretical detention time will be considered when calculating "CT". Pipelines, where very little mixing occurs, will have hydraulic efficiencies that approach 1.0. The following equation can be used to determine the T_{10} through any disinfection section, at any flow rate:

3. $$\mathbf{T_{10}\ (min) = \frac{Hydraulic\ Efficiency\ (e) \times Disinfection\ Section\ Volume\ (gal)}{Flow\ Rate\ (gal/min)}}$$

Though recommended values for hydraulic efficiency are presented in the SWTR Guidance Manual for various treatment processes, the only way to accurately measure hydraulic efficiency is through a tracer test. Once hydraulic efficiency is determined, it can be used to calculate T_{10} at a range of flowrates through a disinfection section (i.e. flocculation/sedimentation basin, clearwell, etc.).

Calculating CT through a WTP

The overall disinfection achieved through a treatment plant and/or a transmission pipeline is simply the sum of the disinfection achieved through the various disinfection sections of the plant. Because the $CT_{required}$ varies according to the water quality characteristics (chlorine residual, pH and temperature), the boundaries of these disinfection sections are typically defined by the points of chemical addition and chemical monitoring (i.e. pH adjustment, chlorine injection/residual monitoring, etc.). The water quality characteristics used to determine the $CT_{required}$ represent the "worst case" conditions (i.e. the highest pH, lowest temperature and highest chlorine residual measured through the disinfection section).

Calculating overall disinfection performance at the Grants Pass WTP, with various points of chlorine injection and/or pH adjustment, is slightly more complex than simply summing the $CT_{achieved}$ through each disinfection section because the $CT_{required}$ varies between sections. Rather, to quantify the overall disinfection performance, the fraction of the overall disinfection requirement (or ratio of the total $CT_{achieved}$ to the $CT_{required}$), or more simply, the log-inactivation achieved through each disinfection section is summed. For example, if 0.2-log inactivation is achieved in the Basins, 0.05-log inactivation is achieved in the Filters, and 0.4-log inactivation is achieved through the clearwell, a total inactivation of 0.65-log was achieved. In this case, the plant would have exceeded the disinfection requirement of 0.5-log inactivation, in compliance with the regulations.

The following equation is used to calculate the log-inactivation achieved through a disinfection section:

4. Log-inactivation = $\dfrac{CT_{achieved}}{CT_{required}}$

Example 1 illustrates the CT calculation for a single disinfection section at the Grants Pass WTP.

EXAMPLE #1: Calculate $CT_{achieved}$ and resulting log-inactivation through the Basins, given the following information:

- T_{10}/T of Basin = 0.28
- Basin Volume (rate-limiting basin) = 214,000 gallons
- Flow in Basin (rate-limiting basin) = 3704 gpm
- Chlorine residual at Basin Effluent = 0.6 mg/L
- Maximum pH = 7.0
- Minimum Water Temperature = 15 deg C

SOLUTION: The $CT_{achieved}$ through the Basin can be calculated according to the following equation:

1. $CT_{achieved}$ (Basin) = Chlorine Residual (Basin effluent) x T_{10} (Basin)

   Where:

   T_{10} (Basin) = hydraulic efficiency x T_{ave}

   Where T_{ave} = Basin Volume (gal) / Flow in Basin (gpm), so

   $$T_{10}\text{ (Basin)} = \frac{\text{hydraulic efficiency x Basin Volume (gal)}}{\text{Flow in Basin (gpm)}}$$

   $$T_{10}\text{ (Basin)} = \frac{\text{0.28 x 214,000 (gallons)}}{\text{3704 (gpm)}}$$

   T_{10} (Basin) = 16 min

   $CT_{achieved}$ (Basin) = 0.6 mg/L x 16 min

   $CT_{achieved}$ (Basin) = 9.7 mg/L –min

2. Using CT Tables from the SWTR Guidance Manual for 1-log inactivation of *Giardia*, $CT_{required}$ at our water quality conditions is **24 mg/L – min**, well above our $CT_{achieved}$. To calculate log-inactivation use a simple ratio:

   $$\textbf{X-log kill} = \frac{\text{9.7 mg/L -min}}{\text{24 mg/L-min}}$$

   or **0.40-log** inactivation of *Giardia*!!!

NOTE: Due to unequal Basin volumes, an option exists in the model to turn the Basins on or off individually to simulate situations involving cleaning or maintenance activities.

Example 2 illustrates a CT calculation for a WTP with variable water quality characteristics.

EXAMPLE #2: Calculate log-inactivation through the Contact Basins and Clearwell, given the following additional information:

- T_{10}/T of Clearwell = 0.51
- Clearwell Volume = 325,000 gallons
- Flow in Clearwell = 11,111 gpm (or 16 mgd total through the plant)
- Chlorine residual at Clearwell Effluent = 0.85 mg/L
- Maximum pH = 8.0
- Minimum Water Temperature = 15 deg C

SOLUTION: The $CT_{achieved}$ through the Basin and Clearwell, combined, can be calculated according to the following equation:

1. Total inactivation (log) = log-inactivation (Basin) + log-inactivation (Clearwell)

We know the log inactivation through the Basin (from Example #1); we must solve for the log-inactivation through the Clearwell. We do this by calculating the $CT_{achieved}$, then solving for the log-inactivation.

$CT_{achieved}$ (Clearwell) = Chlorine Residual (Clearwell effluent) x T_{10} (Clearwell)

Where:

T_{10} (Clearwell) = hydraulic efficiency x T_{ave}

Where T_{ave} = Clearwell Volume (gal) / Flow in Clearwell (gpm), so

$$T_{10}\text{ (Clearwell)} = \frac{\text{hydraulic efficiency x Clearwell Volume (gal)}}{\text{Flow in Clearwell (gpm)}}$$

$$T_{10}\text{ (Clearwell)} = \frac{\text{0.51 x 325,000 (gallons)}}{\text{11,111 (gpm)}}$$

T_{10} (Clearwell) = 14.9 min

$CT_{achieved}$ (Clearwell) = 0.51 mg/L x 14.9 min

$CT_{achieved}$ (Clearwell) = 12.7 mg/L –min

2. Again, using CT Tables from the SWTR Guidance Manual for 1.0-log inactivation of *Giardia*, $CT_{required}$ at our water quality conditions is **36 mg/L – min**. To calculate log-inactivation use a simple ratio:

$$\textbf{X-log kill} = \frac{\text{12.7 mg/L -min}}{\text{36 mg/L-min}}$$

or **0.35-log** inactivation of *Giardia* through the Clearwell.

3. Total inactivation (log) = log-inactivation (Basin) ***(EX 1)*** + log-inactivation (Clearwell)

Total inactivation (log) = 0.40-log + 0.35-log

Total inactivation (log) = 0.75-log

WTP DISINFECTION MODEL

To accurately model the disinfection performance through the Grants Pass WTP, a model was created to perform these calculations. To do so, the plant was divided into the following disinfection sections:

- Contact basins,
- Filters,
- Clearwell,
- Pipeline (not used/optional)

Some components were intentionally excluded in the model due to poor mixing characteristics and/or potential short-circuiting issues that minimize the effective detention time, and render the overall CT achieved through the component inconsequential. These sections include: the Rapid Mix Basin, the channels/pipelines that convey water from the Contact Basins to the filters and the channel that conveys filtered water to the Clearwell.

The model was built in Microsoft Excel, and consists of a series of cross-referenced worksheets. A list of the worksheets, and a brief description follows.

- "User's Guide": contains a link to an embedded PDF copy of this Guide and contact information for the author.
- "Model Input & Results": presents a system schematic and summary of calculated values from the model. User defined variables such as plant flow rate, water quality parameters (pH, chlorine residual, etc…), and operating parameters (number of basins/filters on-line, etc…) can all be input in this worksheet. In most cases, notes and assumptions have been included beneath each of the individual disinfection sections. **In general, input variables are all contained within yellow cells; output variables are contained within red cells.**
- "Performance Summary": provides a graphical representation of *Giardia* log inactivation for the water quality parameters input for different scenarios.
- "CT Look-up Tables": These are the CT tables as transcribed from the SWTR Guidance Manual for 0.5-log inactivation of *Giardia lamblia*. These tables should not be altered.
- "T10 Reference Tables": this worksheet is provided for the user's reference when inputing the T_{10}/T values into the model.

A discussion of the user-input variables on the "Model Input & Results" worksheet (for each of the disinfection sections), as well as the assumptions made when calculating the disinfection performance of each section is discussed below.

General: User-defined variables include plant flow (in mgd) and water temperature (in ^{o}C). These values are referenced and used throughout the spreadsheet when calculating detention times, flow splitting and CT values. Note: the water temperature may increase slightly through the plant, and if desired, temperature should be input separately for each disinfection section.

***Basins*:** Flow to each of the basins has been set-up to use the Grants Pass WTP's flow split configuration. Basins can be turned on and off as needed for modeling purposes. The T_{10}/T values used in the model are based on recommended values presented in the SWTR Guidance Manual.

In addition, the user can also adjust the chlorine residual and pH for the Contact Basins. NOTE: chlorine residual should be based on the residual monitor in the "flume", downstream of the Basins. The volume of the basins (used to calculate the T_{10}) were based on an operator input "Water level in Basin". The basin water level fluctuates with filter water level, which is typically controlled to +/- 0.1 foot, so there will be little change in basin water level.

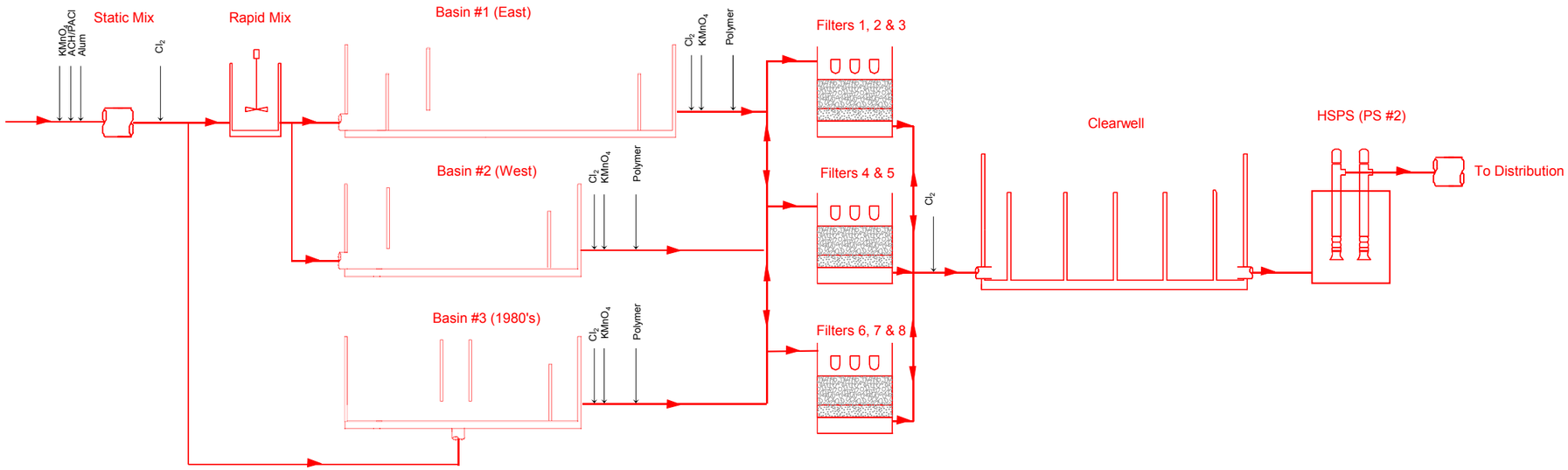
Filters: Flow through the individual filters is calculated by dividing the total plant flow by the number of filters on-line (an user-input variable). In calculating the total volume of the filters, a filter media porosity of 0.5 was used. NOTE: A operator-input variable named "Use Filters for CT?" can be toggled between Y/N, depending on the scenario desired. Typically, the Grants Pass WTP will use the Filters for CT compliance.

***Clearwell*:** For the Clearwell, the user-input variables include: water depth, chlorine residual and pH. Again, water quality characteristics must be measured downstream of the Clearwell (i.e. in the HSPS discharge, for example).

***Pipeline*:** The flow to the transmission pipeline must be input by the user. As with previous disinfection sections, chlorine residual and pH are user-input variables. Again, these water quality values need to be measured at the down-stream side of the transmission pipeline to be considered in the CT calculation. As the Grants Pass WTP does not use the transmission pipeline to meet CT, the input variable are placeholders and the log-inactivation will read "0" for this section. This section of the model is left purely for "what-if" scenarios.

***Totals*:** A summary of the log-inactivation achieved through each of the disinfection sections is presented in this section of the model. Overall totals, including the inactivation achieved through the pipeline (if ever used).

Grants Pass WTP CT Model



Input/Results:

General:

Plant Flow Rate (for CT Calcs)	13500	(gpm)
Plant Flow Rate	19.4	(mgd)
Finished Water Temperature	15.00	(°C)
MAX Design Rate	20	(mgd)
MIN Design Rate	1	(mgd)

Notes:

1. Yellow boxes indicate operator input variables
2. Red boxes indicate calculated values

Notes:

1. Verify flow calculations; include recycle streams where appropriate.

Floc/Sed Basins

General

Flow to Basin #1	6.9	(mgd)
Flow to Basin #2	4.1	(mgd)
Flow to Basins #3	8.4	(mgd)
Basin #1 Online (Y/N)	Y	(mgd)
Basin #2 Online (Y/N)	Y	(mgd)
Basin #3 Online (Y/N)	Y	(mgd)
Water Depth in Basin	14.5	(ft)

Basin #1

MIN Chlorine Residual (effluent)	0.10	(mg/L)
MAX pH (Rapid Mix)	7.50	
Temperature	15.00	(°C)
T_{10}/T	0.44	
Hydraulic Residence Time	119	(min)
T10 (for CT Calculation)	52	(min)
CT Achieved	5.2	(mg/L-min)
Log Inactivation	0.17	(log)

Basin #2:

MIN Chlorine Residual (effluent)	0.10	(mg/L)
MAX pH (Rapid Mix)	7.50	
Temperature	15.00	(°C)
T_{10}/T	0.44	
Hydraulic Residence Time	119	(min)
T10 (for CT Calculation)	52	(min)
CT Achieved	5.2	(mg/L-min)
Log Inactivation	0.17	

Basin #3:

MIN Chlorine Residual (effluent)	0.10	(mg/L)
MAX pH (Rapid Mix)	7.50	
Temperature	15.00	(°C)
T_{10}/T	0.44	
Hydraulic Residence Time	119	(min)
T10 (for CT Calculation)	52	(min)
CT Achieved	5.2	(mg/L-min)
Log Inactivation	0.17	

Notes:

1. T10/T Ratio based on 2012 tracer testing

Anthracite/Sand Filter

General

Number of Filters On-line	8	(no)
Use Filters for CT? (see Note 3)	Y	(Y/N)
Filter Surface Area	311.5	(sf)
Media Depth	30	(in)
MIN Water Depth	6.25	(ft)
Single Filter Volume	11651.66	(gal)

Filters:

MIN Chlorine Residual (effluent)	0.10	(mg/L)
MAX pH (Rapid Mix)	7.50	
Temperature	15.00	(°C)
T_{10}/T	0.44	
Hydraulic Residence Time	6.9	(min)
T10 (for CT Calculation)	3.0	(min)
CT Required	30	(mg/L-min)
CT Achieved	0.3	(mg/L-min)
Log Inactivation	0.01	(log)

Notes:

1. Need to confirm Cl2 & pH sample points to ensure compliance
2. Volume Calcs assume a media porosity of 0.5
3. Filters can be turned off to model other scenarios.

Clearwell:

General

Basin Surface Area	3558	(ft)
Water Depth	13.6	(ft)
Basin Volume	361996.6	(gal)

Clearwell:

MIN Chlorine Residual (effluent)	0.90	(mg/L)
MAX pH (Rapid Mix)	7.50	
Temperature	15.00	(°C)
T_{10}/T	0.53	
Hydraulic Residence Time	27	(min)
T10 (for CT Calculation)	14	(min)
CT Required	30	(mg/L-min)
CT Achieved	12.9	(mg/L-min)
Log Inactivation	0.43	

Notes:

1. T10/T Ratio based on B&V 2003 tracer testing.

Pipeline:

General

Plant Efficiency	0.95	(%)
Pipeline Diameter	36	(in)
Pipe Length	5	(ft)
Pipe Volume	264	(gal)
Pipeline Flow	18.43	(mgd)

Transmission Main to First Customer

Chlorine Residual	0.85	(mg/L)
pH	8.00	
Temperature	15.00	(°C)
T_{10}/T	1.00	
Hydraulic Residence Time	0.0	(min)
T10 (for CT Calculation)	0.0	(min)
CT Required	36	(mg/L-min)
CT Achieved	0.0	(mg/L-min)
Log Inactivation	0.00	

Notes:

1. Pipeline is not used in this calculation and is provided solely for 'what if' scenarios.
2. Pipeline Flow calculated by multipling Plant Flow and Plant Efficiency.

Totals:

In-plant Sub-Total

Process	Log
Floc/Sed Basins	0.17
Clearwell	0.43
Sub-total (In-plant)	***0.61***

24-inch Transmission Main

Pipeline (to First Customer)	0.00
Filter	0.01

Totals:

Total Achieved in Plant	0.61

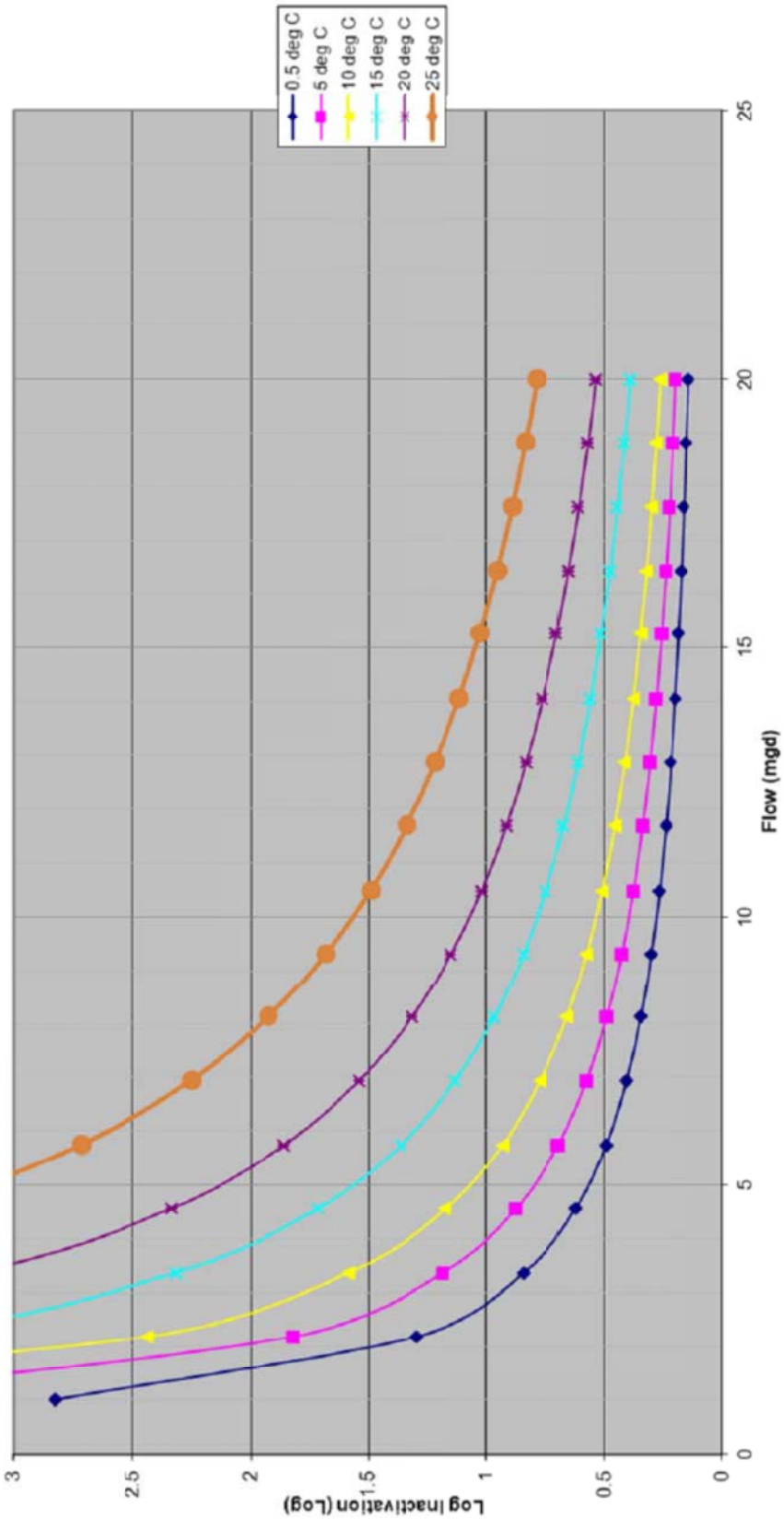


Figure C-2: Log inactivation of *Giardia* through the Clearwell from the CT Model for the Water Quality Parameters Input

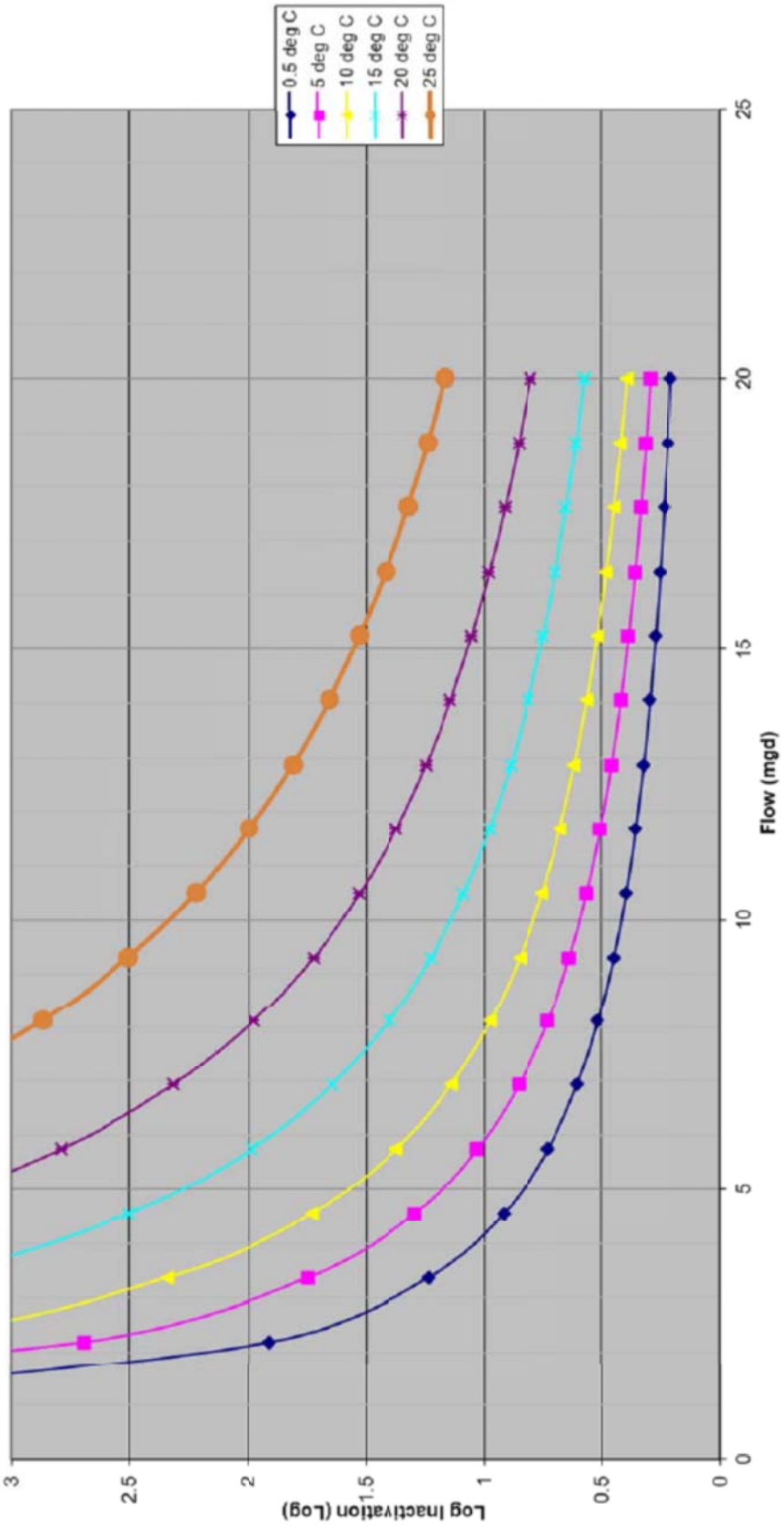
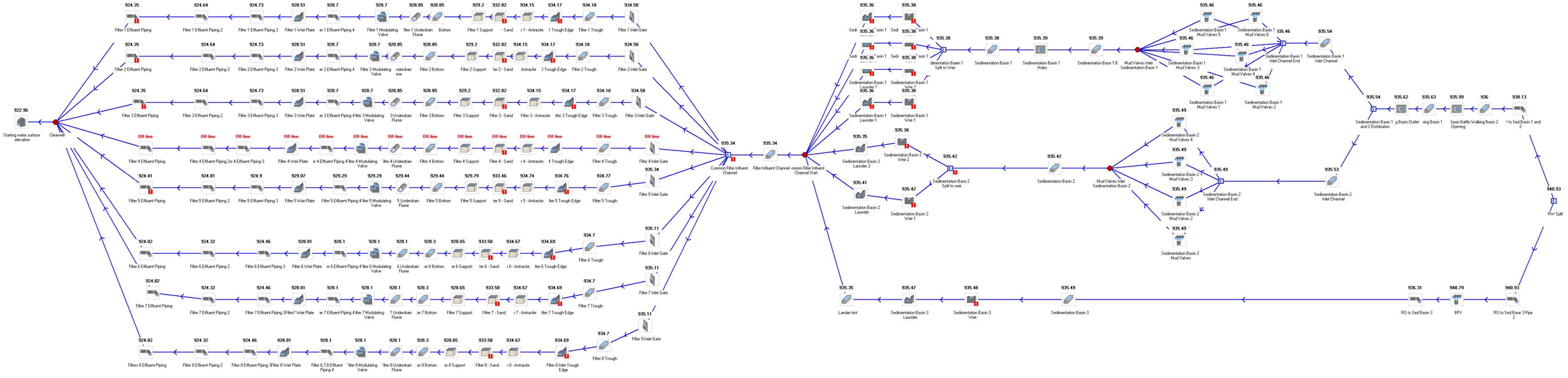


Figure C-3: Log inactivation of *Giardia* through the WTP & Clearwell from the CT Model for the Water Quality Parameters Input

APPENDIX B


922.96
Starting water surface elevation
Cleanwat
924.35
Filter 1 Effluent Piping
924.64
Filter 1 Effluent Piping 2
924.73
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APPENDIX C

MEMORANDUM

TO:	**Jason Canady**	**DATE: 7/15/2011**	
FROM:	**Todd Petrik, P.E., MWH**	**CC:**	**Chris Uber, P.E., MSA**
REVIEWED BY:	**Pete Kreft, P.E., MWH**		**Brian Ginter, P. E., MSA**
	Corie Peterson, P.E., MWH		

SUBJECT: Grants Pass WTP – Inspection of the Clearwell (Inspection Date 5/24/2011)

Background

On May 24, 2011, MWH conducted a follow-up inspection of the clearwell concrete roof and beams in the original 1930's constructed part of the building. A photo log of this visit is attached as Appendix A. The follow-up was conducted to determine a more-defined extent of the concrete deterioration that was first discovered during the March 2, 2011 inspection. As a reference, the March 2, 2011 report and photo log are attached as Appendix B. It should be noted that the March 2, 2011 inspection was focused on the area of the clearwell that was leaking in groundwater, but some other areas of concern were observed then as well. The leak area and general recommendations from the March 2, 2011 inspection are summarized below:

Clearwell Water Intrusion Repair Recommendations

- MWH recommends that the backfill on the exterior of the wall adjacent to the joint between the 1930 and 1950 construction be removed for further inspection of this joint from outside the building.
- Once the cause of the water intrusion is known for sure, then repairs to the leak should proceed as quickly as possible.
- There is more than one way to repair this leak. These are, but are not limited to:
  - An exterior rubber strip sealed on both sides of the crack and covering over the crack to prevent the migration of the groundwater into the building.
  - Injection grouting into the crack
- One thing that should be considered by the City is to chip away the exterior wall concrete down to the 10 ga galv. iron waterstop and remove it altogether. This will help to eliminate any future rusting of this waterstop and help to prevent the rusting of the waterstop and spalling of concrete.

Clearwell General Inspection Repair Recommendations

<u>Ladder Rungs in Each of the Three Clearwell Manhole Access</u>

- None of the ladder rungs are in a state of immediate collapse. Yet, they should be replaced in the near future. It is important to mitigate any further rusting of the ladder rungs back into the concrete wall. New rungs should be installed with a "drill and epoxy" system. The rungs should be either stainless steel or FRP.

Various Concrete Beams in the Roof of the Octagon Clearwell

- None of the beams have deteriorated to a point where the beams are failing. Yet, they should be repaired in the near future. It is important to mitigate any further rusting of the reinforcement. Once the rusting starts, it will travel over time down the length of the entire bar. They are many different options for the repair of these types of deterioration in concrete. These options will be discussed at length with the City Staff at a time closer to put together a repair plan.

The most recent inspection was completed on Tuesday, May 24, 2011 by Todd Petrik and accompanied by the Treatment Plant Superintendent, Jason Canady. A summary of this visit and recommendations that resulted from it are listed below.

Clearwell - General Structural Inspection

Note: All of the photos referenced in the following text below are included in the attached annotated site photo log, attached as Appendix A.

Generally, the concrete floors, walls, columns, beams, and undersides of the top decks are in good condition. However, a more aggressive approach was taken during the most recent inspection to determine how far-reaching the spalling concrete and corroding reinforcement is that was first encountered during the March 3 inspection. For those areas that do have issues, the extent of the deterioration is worse than originally determined. If certain areas are not repaired, damage could continue to spread and threaten the structural stability and integrity of the clearwell. In turn, this could potentially threaten the ability of the Water Treatment Facility to effectively disinfect water and deliver safe drinking water to the citizens of Grants Pass.

Identification of Various Concrete Beams in the Roof of the Octagon Clearwell

There are four interior concrete columns in the clearwell that divide the roof/beams into nine different panels. Running in the North/South direction, there are north, middle and south beams. Running in the East/West direction, there are east, middle and west beams. These beams and the exterior walls of the clearwell define the 9 different panels. Reference the attached sketch presented as **Figure 1**. The summary below makes use of the labeling defined above and shown in **Figure 1**.

Common/Typical General Concrete Repair Methods

One common, and often-used, method for the repair of concrete structural elements is to use a bonded fiberglass reinforced plastic (FRP) system. This practice is governed by ACI 440.2R and is widely used throughout the engineering community. For this report, this process will be referred to as FRP Repair.

Repair Priority

MWH has determined that the repairs presented below fall into two different Priority categories.

Priority 1: Denoted by “P1”, this is the higher of the two levels. These repairs should be made in the next Low-Water demand period. These repairs should be given a higher priority to prevent the conditions from growing worse which could lead to a failure of the structural systems in the clearwell ceiling.

Priority 2: Denoted by “P2”, this is the lower of the two levels. These issues have much less of an impact on the structural system in the clearwell ceiling. Not completing these repairs will result in worsening of the condition, yet it is not urgent that these repairs take place in the next Low-water demand period,

1. **West-South Beam (Photos 0070 to 0074)**

The bottom-west corner of this beam has rock pockets that extend south about 30-inches in length from the interior column. There is no visible reinforcement. It appears that the concrete was never consolidated in this corner during the initial construction.

P2-Repair: Sand blast clean and fill the pockets with a non-shrink epoxy grout.

2. **South-West Beam (Photos 0075 to 0080)**

The bottom-south corner of this beam has rock pockets that extend from the west wall for about half the length of the beam to the east. There is no visible reinforcement. It appears that the concrete was never consolidated in this corner during the initial construction. On the south side of this beam, there is an abandoned steel floor drain. It is rusting and starting to spall the concrete around it.

P2-Rock Pocket Repair: Sand blast clean and fill the pockets with a non-shrink epoxy grout.

P2-Pipe Repair: Cut off the drain pipe and burn it back up into the concrete a minimum of 1-1/2 inches. Grout over the hole with a non-shrink epoxy grout.

3. **West-Middle Beam (Photos 0082 to 0085)**

The bottom-west corner of this beam from the north column to 48-inches south has significant concrete spalling and deteriorated reinforcing bars. The east side of the beam has a short, 12 to 16-inch section of rock pockets in the mid-span of the lower corner. On the east side of this beam, there is an abandoned steel floor drain. It is rusting and starting to spall the concrete around it.

P1-Reinforcement Repair: Chip back the concrete until clean, bright reinforcement is encountered. Remove the rusted reinforcement. Build back up the deteriorated concrete beam with a non-shrink grout and the use of the FRP repair method to bring back the integrity of this beam.

P2-Rock Pocket Repair: Sand blast clean and fill the pockets with a non-shrink epoxy grout.

P2-Pipe Repair: Cut off the drain pipe and burn it back up into the concrete a minimum of 1-1/2 inches. Grout over the hole with a non-shrink epoxy grout.

4. **South-Middle Beam (Photos 0089 to 0096)**

At the west end of this beam and in the center of the beam, there is a hole from unconsolidated concrete that has resulted in exposed reinforcing bars. The two exposed bars are rusting. The rusting of these bars is causing further spalling of the concrete. Along the north side of the beam, there is a rock pocket that extends for about a 5 foot length.

P1-Reinforcement Repair: Chip back the concrete until clean, bright reinforcement is encountered. Remove the rusted reinforcement. Build back up the deteriorated concrete beam with a non-shrink grout and the use of the repair method to bring back the integrity of this beam.

P2-Rock Pocket Repair: Sand blast clean and fill the pockets with a non-shrink epoxy grout.

5. **East-Middle Beam (Photos 0097 to 0103)**

At the bottom west corner of the beam and for the entire length of the beam, the concrete has spalled off and exposed the reinforcing bar(s). Also, along the east side of the beam, there is a short 6 to 8-inch long section of spalled concrete and exposed reinforcement.

P1-Reinforcement Repair: For the entire length of this beam, chip back the concrete until clean, bright reinforcement is encountered. Remove the rusted reinforcement. Build back up the deteriorated concrete beam with a non-shrink grout and the use of the FRP repair method to bring back the integrity of this beam.

6. Middle-East Ceiling Panel (Photos 0104 to 0110)

There is a 54-foot long section of reinforcement that has rusted and spalled the concrete around it. There is a 2-inch deep hole in the bottom of the floor from a deteriorated wood block. Also, there is an abandoned steel floor drain pipe that is rusting and starting to spall the concrete around it.

P1-Reinforcement Repair: Chip back the concrete until clean, bright reinforcement is encountered. Remove the rusted reinforcement. Build back up the deteriorated concrete floor with a non-shrink grout and the use of the FRP repair method to bring back the integrity of this floor.

P1-Hole Repair: Chip back the concrete until clean, bright reinforcement is encountered. Remove the rusted reinforcement. Build back up the deteriorated concrete floor with a non-shrink grout and the use of the RFP repair method to bring back the integrity of this floor.

P2-Pipe Repair: Cut off the drain pipe and burn it back up into the concrete a minimum of 1-1/2 inches. Grout over the hole with a non-shrink epoxy grout.

7. East-South Beam (Photos 0111 to 0116)

Along the west and east sides of the beam, there are various rock pockets. There is no visible rusted reinforcement showing. Near the center column. there is an abandoned pipe stuffed with some type of insulation. The pipe is rusting and spalling the concrete around it.

P2-Rock Pocket Repair: Sand blast clean and fill the pockets with a non-shrink epoxy grout.

P2-Pipe Repair: Cut off the drain pipe and burn it back up into the concrete a minimum of 1-1/2 inches. Grout over the hole with a non-shrink epoxy grout.

8. South-East Beam (Photos 0117 to 0120)

There is no spalling of the concrete or any visible rusting reinforcement on either side or bottom of the beam.

Repair: None required

9. South-East Ceiling Panel (Photos 0121 to 0124)

In the underside of the floor, there is a large circular patch that appears to have the hole filled at two different times. The outer patch is starting to deteriorate and the older concrete is spalling off. There is no visible rusting reinforcement.

P2-Patch Repair: Chip out the old concrete to sound, solid concrete. Sandblast the hole clean. Place new non-shrink grout into the hole. If the hole becomes over 12-inches in diameter, there may be a need to drill and epoxy in small reinforcing bars to hold the new grout into place.

10. Clearwell Access hole in East wall (Photos 125 to 131)

In order to expand the clearwell to the East, a hole was cut in the wall. When the hole was cut, it appears that reinforcing bars were left exposed. These reinforcing bars are now rusting and spalling the concrete around them. While there is no real structural significance to this, it is worth cleaning out the old bars and patching up the edges of the hole.

P1-Repair: Chip the concrete back to expose clean bright reinforcement. Cut off all rusted reinforcement and burn the bars back into the solid concrete a minimum of 1-1/2 inches. Drill in new, short #4 dowels around the edge of the opening. Form a new opening and cast in new non-shrink grout.

11. North-West Beam (Photos 0132 to 0136)

On the north side of the beam and on the west end, the bottom corner has spalled off and rusted reinforcement is visible.

P1-Reinforcement Repair: For the entire length of this beam, chip back the concrete until clean, bright reinforcement is encountered. Remove the rusted reinforcement. Build back up the deteriorated concrete beam with a non-shrink grout and the use of the FRP repair method to bring back the integrity of this beam.

12. North-West Ceiling Panel (Photo 138)

There is a small hole that has spalling concrete around it. There is no visible rusting of reinforcement.

P2-Hole Repair: Chip back the concrete until clean, bright reinforcement is encountered. Remove the rusted reinforcement. Build back up the deteriorated concrete floor with a non-shrink grout and the use of the RFP repair method to bring back the integrity of this floor.

13. West-North Beam (Photos 139 to 141)

There is no spalling of the concrete or any visible rusting reinforcement on either side or bottom of the beam. There are a few small rock pockets along the length of the beam.

P2-Rock Pocket Repair: Sand blast clean and fill the pockets with a non-shrink epoxy grout.

14. North-Middle Beam (Photos 142 to 148)

There is no spalling of the concrete or any visible rusting reinforcement on either side or bottom of the beam. There are a few small rock pockets along the length of the beam. At the west end of the beam, adjacent to the large patch in the ceiling, there is a small hole in the corner where the beam meets the ceiling. There is a 2 to 3-inch long piece of reinforcement that is visible. This reinforcement is not rusting and the concrete is not spalling.

P2-Rock Pocket Repair: Sand blast clean and fill the pockets with a non-shrink epoxy grout.

P2-Small Hole Repair: Sand blast clean and fill the hole with a non-shrink epoxy grout.

15. North-Middle Ceiling Panel (Photos 149 to 153)

Up tight against the north wall of the clearwell, there are two exposed bars that are rusting and starting to spall the concrete. The west bar is exposed for about 6 to 8-inches. The East bar is exposed for

about 14 to 16-inches. And there is an abandoned steel floor drain pipe that is rusting and starting to spall the concrete around it.

P1-Reinforcement Repair: Chip back the concrete until clean, bright reinforcement is encountered. Remove the rusted reinforcement. Build back up the deteriorated concrete floor with a non-shrink grout and the use of the FRP repair method to bring back the integrity of this floor.

P2-Pipe Repair: Cut off the drain pipe and burn it back up into the concrete a minimum of 1-1/2 inches. Grout over the hole with a non-shrink epoxy grout.

16. North-East Beam (Photos 154 to 159)

There is one hole in the middle of the underside of the beam. It is about 2-inches in diameter and about 2-inches deep. There is rusting reinforcement visible in the hole. The rest of the beam is in good conditions.

P2-Hole Repair: Chip back the concrete until clean, bright reinforcement is encountered. Remove the rusted reinforcement. Build back up the deteriorated concrete floor with a non-shrink grout and the use of the FRP repair method to bring back the integrity of this floor.

17. North-East Beam (Photos 0160 to 0165)

On the east end of the beam, along the bottom south corner for 3 feet, the concrete is spalling off and rusted reinforcement is visible. Also, there is a 12- to 14-inch long section of spalling concrete and rusting reinforcement in the center of the beam.

P1-Reinforcement Repair: For the entire length of this beam, chip back the concrete until clean, bright reinforcement is encountered. Remove the rusted reinforcement. Build back up the deteriorated concrete beam with a non-shrink grout and the use of the FRP repair method to bring back the integrity of this beam.

18. North-East Ceiling Panel (Photos 0166 to 0169)

There are three smaller holes in the ceiling, each having a small section of rusting reinforcement visible in the hole. There are also several small rock pockets with no visible reinforcement.

P1-Reinforcement Repair: Chip back the concrete until clean, bright reinforcement is encountered. Remove the rusted reinforcement. Build back up the deteriorated concrete floor with a non-shrink grout and the use of the FRP repair method to bring back the integrity of this floor.

P2-Rock Pocket Repair: Sand blast clean and fill the pockets with a non-shrink epoxy grout.

19. Clearwell East of the 1930 octagon (Photos 0170 to 0194)

There did not appear to be any visible spalling of concrete or exposed and rusting reinforcement.

Repairs: None required.

20. Clearwell west opf the 1930 Octagon (Photos 195 to 198)

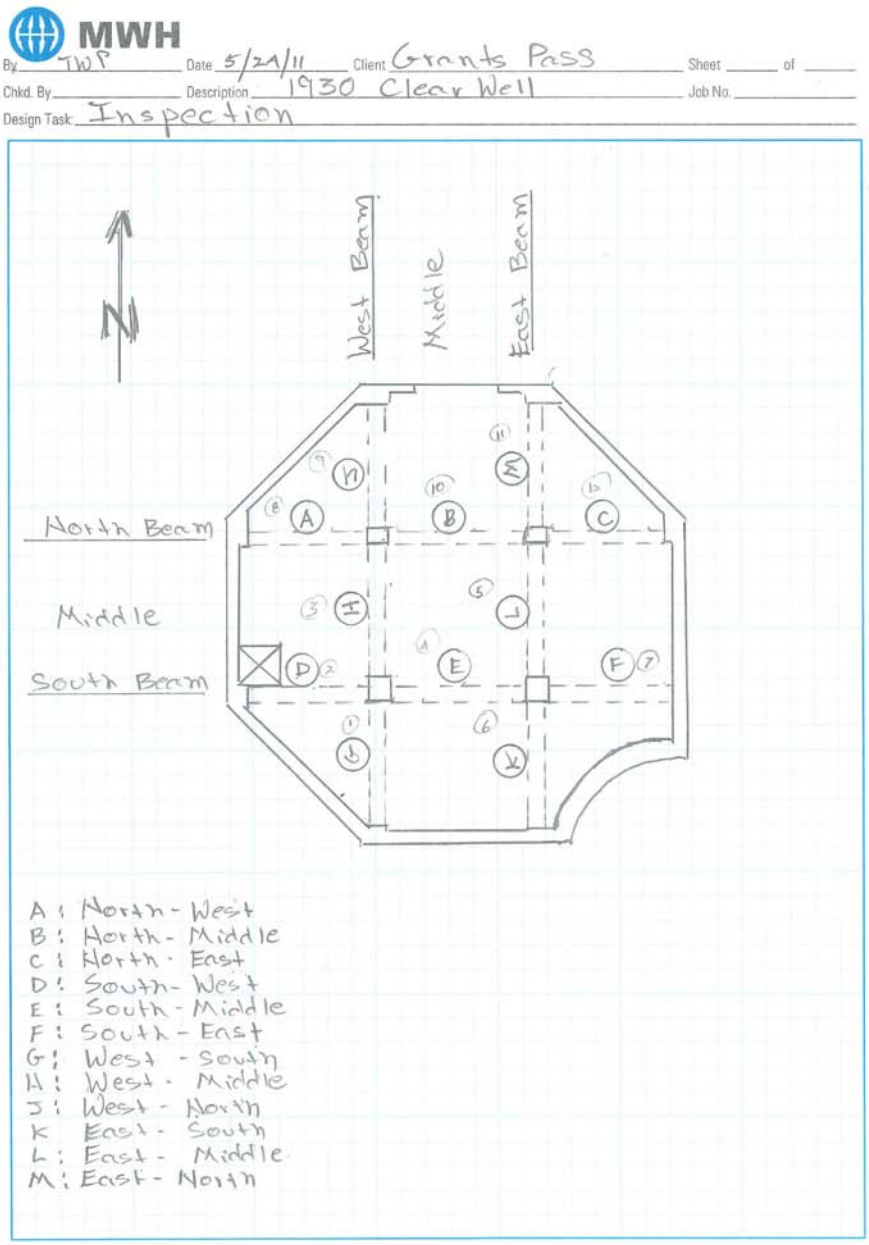
There did not appear to be any visible spalling of concrete or exposed and rusting reinforcement.

Repairs: None required.

Summary

MWH strongly recommends that the P1 level repairs presented in this memo occur during the next low demand period of operation as the repairs will most likely require the clearwell to be taken out of service and dewatered for a period of time. Should the City choose to move forward with repairs, a detailed set of repair plans and specification that can be used for construction should be prepared. In addition, during the next low demand period, the clearwell is slated to be dewatered for construction related to the installation of a new redundant filter backwash pump. Combining these projects may provide significant savings to the City in mobilization costs of similar contractor types, and common construction periods for both categories of work while the clearwell is dewatered.

During any of the repairs stated above, the floor deck and possibly the adjacent beams are to be shored to limit the possibility of the floor deflecting.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By TWP
Date 5/24/11
Client Grants Pass
Sheet of
Chkd. By
Description 1930 Clear Well
Job No.
Design Task: Inspection
West Beam
Middle
East Beam
North Beam
Middle
South Beam
A : North - West
B : North - Middle
C : North - East
D : South - West
E : South - Middle
F : South - East
G : West - South
H : West - Middle
J : West - North
K East - South
L : East - Middle
M : East - North

Figure 1

Appendix A

City of Grants Pass
Water Filtration Plant
821 SE M Street
Grants Pass OR 97526

Inspection Date: **May 24, 2011**
Inspected By: **Todd Petrik**
Clearwell

West-South Beam
Photos 00700 – 0074

Photo 0070

- The concrete is Sound overall
- Rock pockets on west side. 30" in length.

Photo 0071
Showing the rock Pockets

Photo 0072
Showing the rock Pockets

Photo 0073

Photo 0074

South-West Beam
Photos 0075 – 0080

Photo 0075

- The concrete is Sound overall
- Rock pockets on south side

At west end of the beam, there is a drain in slab that is rusting and needs to be cut back and slab bottom patched.

Photo 0076

Photo 0077
Showing the rusting floor drain

Photo 0078
Showing the rusting floor drain

Photo 0079
Showing the rusting floor drain

Photo 0080

South Clearwell Wall
Photo 0081

Rock pocket in south wall

West-Middle Beam
Photos 0082 – 0085

Photo 0082

- West side beam at the lower corner has 48” of spalling concrete and exposed reinforcement.
- Scraped the Reinforcing bar & lost about 50% of the area of reinforcement..
- East side is okay

Photo 0083

No real spalling of concrete. Burn back pipe and patch.

Photo 0084

- West side beam at the lower corner has 48” of spalling concrete and exposed reinforcement.

Photo 0085

- West side beam at the lower corner has 48” of spalling concrete and exposed reinforcement.

Photo 0086
Exposed steel pipe that is rusting

Photo 0087

East side rock pocket

Photo 0088

Pipe in ceiling & reinforcement in corner

South-Middle Beam
Photos 0089 – 0096
Photo 0089

- At the West end of the beam and in the middle of the beam width there is a deep rock pocket – with rusted reinforcement showing. 16” wide at this part of beam.
- East end solid
- The clearwell ceiling is okay in this area
- North side of beam has rock pockets for a length of 5’-0”

Photo 0090

- North side of beam has rock pockets for a length of 5’-0”

Photo 0091

- Rock pockets and exposed reinforcement

Photo 0092

Photo 0093

- North side of beam has rock pockets for a length of 5’-0”

Photo 0094

- Rock pockets and exposed reinforcement

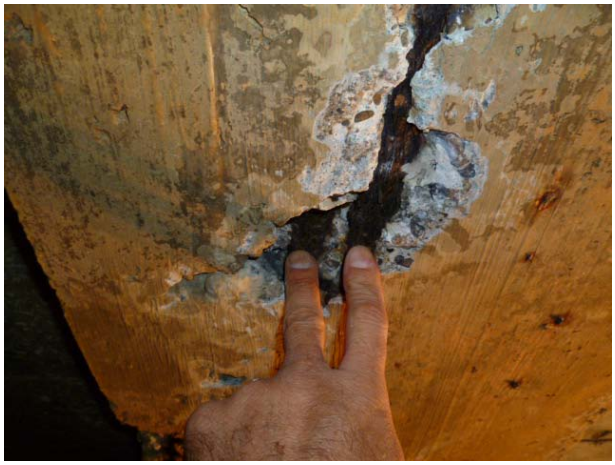


Photo 0095

- Rock pockets and exposed reinforcement

Photo 0096

East-Middle Beam
Photos 0097 – 0103

Photo 0097

- The entire bottom corner on the west side of the beam up to ceiling has spalled off.
- 1 bar is exposed the entire length of spall
- 2 bars are exposed at the center of beam
- At the East side of the beam at north end, there is also spalling concrete and exposed reinforcement.

Photo 0098

- Showing a close up of the spalled concrete and exposed bar on the west side of the beam.

Photo 0099

- Showing a close up of the spalled concrete and exposed bar on the west side of the beam.

Photo 0100

- Showing a close up of the spalled concrete and exposed bar on the west side of the beam.

Photo 0101

- Showing the spalled concrete at the center of the beam

Photo 0102

Photo 0103

- Showing the spalled concrete at the center of the beam

Middle-East Ceiling Panel
Photos 0104 to 0110

Photo 0104

- Showing a patch and a hole in the ceiling

Photo 0105

Photo 0106
Reinforcement bar with no cover has spalled the concrete for about a 5’-0” length

Photo 0107

Photo 0108

- 2-inch deep hole with rotting wood up inside of it.

Photo 0109

- 2-inch deep hole with rotting wood up inside of it.

Photo 0110

- Abandoned steel floor drain pipe that is rusting.

East-south Beam
Photos 0111 – 0116

Photo 0111

- There are rock pockets on the west and east sides of the beam
- There is no visible rusting reinforcement in the rock pockets
- Near the center column There is an abandoned steel pipe stuffed with installation and about 2-inches deep of spalled of concrete

Photo 0112

Photo 0113

- Showing rock pockets

Photo 0114

- Showing the abandoned steel pipe with insulation

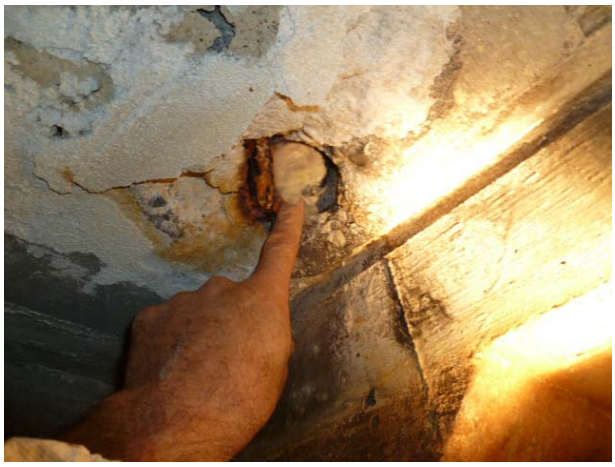


Photo 0115

- Showing the abandoned steel pipe with insulation

Photo 0116

- Showing the abandoned steel pipe with insulation

South-East Beam
Photos 0117 – 0120

Photo 0117

- The concrete is solid on all sides and the bottom of the beam
- No spalling of concrete
- No visible reinforcement

Photo 0118

Photo 0119

Photo 0120

South-East Ceiling Panel
Photos 0121 – 0124

Photo 0121

- There is a circular inside patch and an irregular patch around the outside of the circle.
- The outside patch is spalling and revealing rusting reinforcement

Photo 0122

Photo 0123

Photo 0124

Wall at the Clearwell southeast corner
Photos 0125 - 0127

Photo 0125

- Showing a short section of rusting reinforcement at the face of the wall

Photo 0126

Photo 0127

- There is a small length of exposed reinforcement and spalled concrete

Clearwell access hole in East wall (Photos 125 to 131)

Photo 0128

- Hole cut in East Wall for water flow and access
- The reinforcement around the edge of the opening is rusting and spalling of section of concrete.
- The wall is about 4-1/4 inches wide

Photo 0129

- Showing the edge of the opening

Photo 0130

- Showing the edge of the opening

Photo 0131

North-West Beam
Photos 0132 - 0136

Photo 0132

- At the North side at west end the entire corner has spalled off
- The corner reinforcing bar is exposed and the middle bar is exposed for about 4’-0”

Photo 0133

Photo 0134

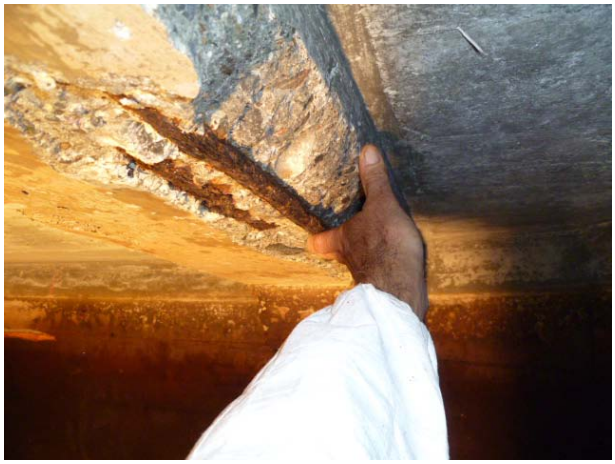


Photo 0135

- Showing exposed reinforcing bars

Photo 0136

North-West Ceiling Panel
Photos 0137 – 0138

Photo 0137

- Cone-tie hole needs to be patched

Photo 0138

- Cone-tie hole needs to be patched

West-North Beam
Photos 0139 – 0141

Photo 0139

- The concrete is solid on both sides and the bottom of the beam
- No spalling
- No rusting reinforcement
- Small rock pockets

Photo 0140

Photo 0141

North-Middle Beam
Photos 0142 - 146

Photo 0142

- The concrete is solid on both sides and the bottom of the beam
- No spalling
- No rusting reinforcement
- Small rock pockets

Photo 0143

Photo 0144

Photo 0145

- Showing a small hole at the west end and it is about 1-1/2 inches deep with a short piece of
- Reinforcement exposed but not rusted

Photo 0146

North-Middle Ceiling Panel
(Photos 147 to 153)

Photo 0147

- There are two patched holes where it appears there used to be pump cans. Both of the patches are in good condition

Photo 0148

Photo 0149

- Against north wall are 2 spalling and rusting reinforcement bars.

Photo 0150

Photo 0151

- The West bar is rusting and spalling concrete for about 6-inches from the wall

Photo 0152

- The East bar is rusting and spalling concrete for about 14-inches from the wall

Photo 0153

- Shows an abandoned pipe. The concrete around it is good but the end of the pipe is rusting

North-East Beam
Photos 0154 - 0159

Photo 0154

- There is one main hole in the middle of the beam about 1 to 2-inches deep
- In the hole, the reinforcement is exposed and rusting
- All other parts of the beam are solid

Photo 0155

Photo 0156

Photo 0157

- Showing the hole in the beam

Photo 0158

Photo 0159

- Showing at the south end of beam on the west side there is a small rock pocket

North-East Beam
Photos 0160 – 0165

Photo 0160

- At the east end of the beam on the south side of beam there is spalling concrete and reinforcement is rusting
- There is also spalling at the center of beam about 3’-0” from the east end
- At the west end of the beam the concrete is solid

Photo 0161

Photo 0162

- Showing the spalling concrete and rusting reinforcement at the east end of the beam

Photo 0163

Photo 0164

Photo 0165

North-East Ceiling Panel
Photos 0166 - 0169

Photo 0166

- There is a concrete cone tie hole 2-inches deep and about 1-1/2 inches in diameter at the concrete surface
- There are also 2 other small reinforcement or tie rusting spots
- And there are a few smal rock pockets

Photo 0167

- Showing the small reinforcement rusting

Photo 0168

Photo 0169

Clearwell East of the 1930 octagon (Photos 0170 to 0194)

For all of this area, there is no visible spalling of concrete or rusting reinforcement

Photo 0170

Photo 0171

Photo 0172

Photo 0173

Photo 0174

Photo 0175

Photo 0176

Photo 0177

Photo 0178

Photo 0179

Photo 0180

Photo 0181

Photo 0182

Photo 0183

Photo 0184

Photo 0185

Photo 0186

Photo 0187

Photo 0188

Photo 0189

Photo 0190

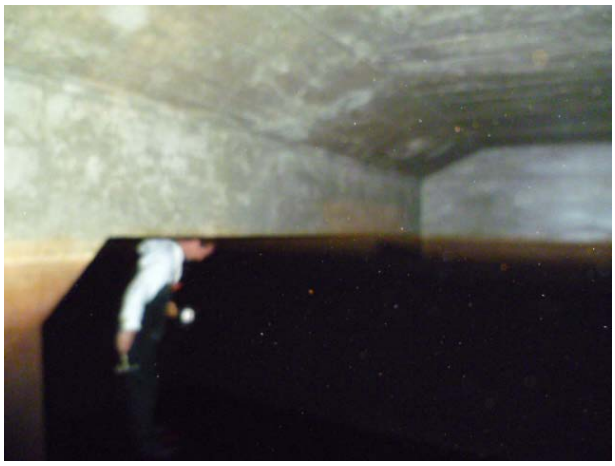


Photo 0191

Photo 0192

Photo 0193

Photo 0194

Clearwell West of the 1930 octagon (Photos 0195 to 0198)

For all of this area, there is no visible spalling of concrete or rusting reinforcement

Photo 0195

Photo 0196

Photo 0197

Photo 0198

Appendix B

MEMORANDUM

To:	**Jason Canady, City of Grants Pass**
From:	**Todd Petrik, MWH**
Date:	**3/15/2011 – Inspection Date**
Subject:	**Grants Pass, Inspection of the Clearwell**

Background

The staff at the Water Treatment plant in Grants Pass contacted MWH to come inspect water intrusion into the interior spaces of the plant. Plant staff stated that it appear that the water intrusion is occurring at a joint that is between the original 1930 construction and the West Clearwell constructed in the 1950s.

In summary, the structural inspection was to review the following items at the plant:

- The current condition of the water intrusion in the space that is above the clearwell in the pipe gallery west of the Octagon lobby. This is dry space.
- The current condition of the water instruction in the water channel down in the clearwell west of the Octagon Lobby. This is wet space.
- Since the entire clearwell was able to be de-watered, a walkthrough of all three spaces was completed.

Clearwell Water Intrusion Inspection Summary

Each of the spaces stated above are below the final grade that is exterior to the building and adjacent to where the water intrusion is occurring.

(All of the photos referenced below are included in the attached site inspection photo log.)

<u>Dry Space Above the Clearwell</u>

1. Photos 1 through 3 show the area of water intrusion from the exterior of the building.
2. Photos 4 and 5 show the inside wall of the 1930s constructed Octagon. There was no indication of water instruction into this space. It appears that the wall of the original 1930 building is holding up to any water intrusion.
3. Photos 6 through 8 shows where the water intrusion is occurring into the space constructed in the 1950s. The conclusion is that the water is coming in through the joint between the exterior wall of the 1930 building and the exterior wall of the 1950 building. The construction detail of this joint can be found in the 1950 expansion drawings in the upper right corner of Sheet No. 5 of 13.

<u>Wet Space Down in the Clearwell</u>

1. Photos 9 through 11 show the joint between the manhole access into the clearwell and the top slab. The water that is coming in from outside has also found its way into this joint. This water is rusting the reinforcement steel in the joint and spalling the concrete. This spalling and rusting is only evident on the East side of the manhole access, which, is adjacent to the vertical crack in the exterior wall shown in Photos 6 through 8.

2. While the channel below the top deck, down in the clearwell is wet, the top part of the channel is above the water line and remains dry. The concrete here also looks dry. This is the case except in the corner at the joint between the 1930 and 1950 construction. Reference photo 12. This picture shows a white effloresce.

Clearwell Water Intrusion Repair Recommendations

- MWH recommends that the back fill on the exterior of the wall adjacent to the joint between the 1930 and 1950 construction be removed for further inspection of this joint from outside the building.
- The most likely outcome will reveal that the water intrusion is starting in this exterior fill and migrating through the joint between the 1930 and 1950 buildings. This would mean that the seal shown in the 1950 drawing detail on sheet 5 of 13 has deteriorated. This seal, or waterstop, is called out in this detail as a "10ga galv. Iron – caulked into a saw cut slot and grouted into place with lead or cement".
- Once the cause of the water intrusion is know for sure, then repairs to the leak should proceed as quickly as possible.
- There is more than one way to repair this leak. These are, but are not limited to:
  - An exterior rubber strip sealed on both sides of the crack and covering over the crack to prevent the migration of the groundwater into the building.
  - Injection grouting into the crack
- One thing that should be considered by the City is to chip away the exterior wall concrete down to the 10 ga galv. iron waterstop and remove it altogether. This will help to eliminate any future rusting of this waterstop and help to prevent the rusting of the waterstop and spalling of concrete.

Clearwell, General Inspection Summary

In general, all of the concrete floors, walls, columns, beams, undersides of the top decks and the interior baffle walls are in good condition. There are a total of three different chambers that make up the complete clearwell and these three chambers also serve as the chlorine contact basin. The flow of the water goes in the direction as follows:

- Starts in the channel on the North side of the west clearwell constructed in the 1950 addition
- Through a pipe that passes through the clearwell constructed in the original 1930 building
- Into the east clearwell that was constructed in the 1980 building.
- Through baffle walls in the east well, into the 1930 well and finally into the west well.

There are several areas found during the inspection that are of concern and a repair for these issues should be put forth soon.

(All of the photos referenced below are included in the attached site photo log.)

Ladder Rungs in Each of the Three Clearwell Manhole Access

1. Photos 13 through 19 show the ladder rungs in the three clearwell access manholes. Most all of the ladder rungs are in poor shape and are in need of replacement sometime in the near future.

Various Concrete Beams in the Roof of the Octagon Clearwell

2. Photos 20, 21 and 22 show the corner of one of the clearwell floor beams. This floor beam (floor to the dry room above and roof to the wet clearwell space below) occurs south of the NW column in the octagon well. The reinforcement rusting and concrete spalling occurs on the bottom west corner of the beam.
3. Photos 23 through 26 show the corner of one of the other clearwell floor beams. This floor beam (floor to the dry room above and roof to the wet clearwell space below) occurs west of the NW column in the octagon clearwell. The reinforcement rusting and concrete spalling occurs on the bottom north corner of the beam.
4. Photo 27 shows a spot in the center of a third beam. This beam occurs north of the NE column in the octagon clearwell. It appears that there was a wood block left in place during the construction. The wood block has rotted and exposed the reinforcement. Then, the reinforcement rusted, causing concrete around the hole to start spalling away from the beam.

Clearwell General Inspection Repair Recommendations

Ladder Rungs in Each of the Three Clearwell Manhole Access

- None of the ladder rungs are in a state of immediate collapse. Yet they should be replaced in the near future. It is important to mitigate any further rusting of the ladder rungs back into the concrete wall. New rungs should be installed with a “drill and epoxy” system. The rungs should be either stainless steel or FRP.

Various Concrete Beams in the Roof of the Octagon Clearwell

- None of the beams have deteriorated to a point where the beams are failing. Yet, they should be repaired in the near future. It is important to mitigate any further rusting of the reinforcement. Once the rusting starts, it will travel over time down the length of the entire bar. They are many different options for the repair of these types of deterioration in concrete. These options will be discussed at length with the City Staff at a time closer to put together a repair plan.

City of Grants Pass
Water Filtration Plant
821 SE M Street
Grants Pass OR 97526

Inspection Date: **March 3, 2011**
Inspected By: **Todd Petrik**
Clearwell

<u>Inspection Photos of the Water Intrusion</u>

Photo 1
Overview of the interface of the 1930 (left side of photo) and 1950 (right side of photo)

The water intrusion is occurring directly behind the water fountain through a crack that is visable from the exterior of the building.

Photo 2

Close up of the interface between the 1930s and 1950 construction.

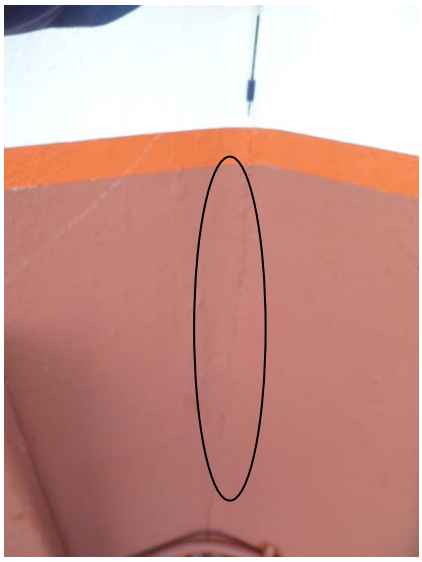


Photo 3

Close in view showing the crack between the 1930s and 1950s construction. This crack was visible from the eave of the roof line clear down to the walk that was sitting on grade

Photo 4

Photos 4 & 5 showing the interface between the 1930 and 1950 construction from inside the Octagon structure, or the 1930s construction.

Photo 5

Photo 6

Photos 6, 7 and 8 show the interface from inside the 1950 construction

Photo 7

Water is coming in from outside in the location directly in the corner where the dark stain is traveling up the joint.

Photo 8

Photo 9

This photo shows the interface, or joint, between the top of the clearwell slab and the tall curb around the manhole access.

Photo 10

Photo11

Photo 12

This photo shows the underside of the top slab interfacing with exterior wall of the 1930 construction (right hand wall) and the exterior wall of the 1950 construction (left hand wall). The white in the picture is staining from the water intrusion that is coming through this joint from outside the building.

Photos for the General Inspection of all three Clearwells

Photo 13

This photo and photo 14 shows the ladder rungs in the manhole access that enters into the channel along the north side of the west clearwell. These ladder rungs were installed in the 1950 construction.

Photo 14
This photo is a close up of the joint.

Photo 15

Photos 15, 16 and 17 show the ladder rungs that enter into the east clearwell.

These ladder rungs were installed in the 1980 construction.

Photo 16

Photo 17

Photo 18

This photo and photo 19 show the ladder rungs extending out of the access in the center, Octagon clearwell. These rungs were most likely installed in the 1930 construction.

Photo 19

Photo 20

Photos 20, 21 and 22 show a corner of one of the beams in the octagon clearwell. This beam is located south of the NW column in the octagon clearwell. The spalling is occurring on the west bottom corner of this beam.

Photo 21

Photo 22

Photo 23

Photos 23, 24, 25 and 26 shows a corner of one of the other beams in the octagon clearwell. This beam is located west of the NW column in the octagon clearwell. The spalling is occurring on the north bottom corner of this beam.

Photo 24

Photo 25

Photo 26

.

Photo 27

This photo shows a spot in the center of the beam where the reinforcement is rusting and the concrete is spalling. This beam is located north of the NE column in the octagon clearwell. It appears that there was a wood block left in place during the construction. The wood has rotted and exposed the reinforcement. Then the reinforcement rusted causing concrete around the hole to start spalling away from the beam.

APPENDIX D

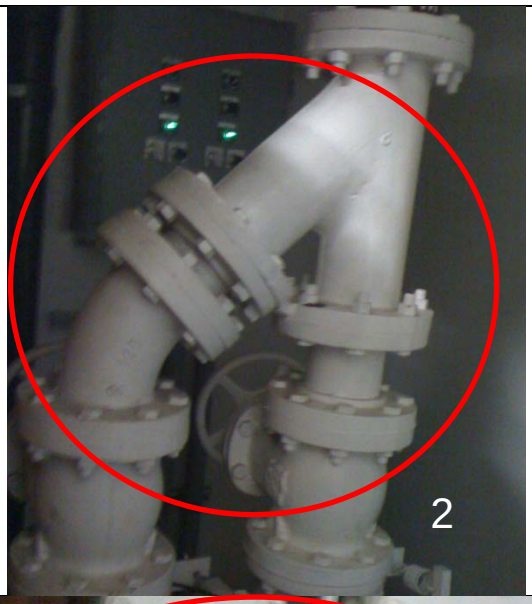
APPENDIX D
PHOTOGRAPH LOG AND PLANT EQUIPMENT INVENTORY

Photo Log - Table of Contents

Break Areas

Limited area and space in break room and break areas.

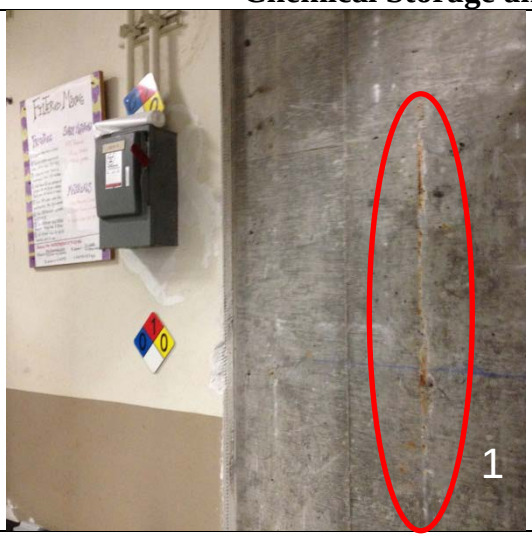
Panel board located in break room.



1. Cosmetic damage to tiling and general wear due to age.

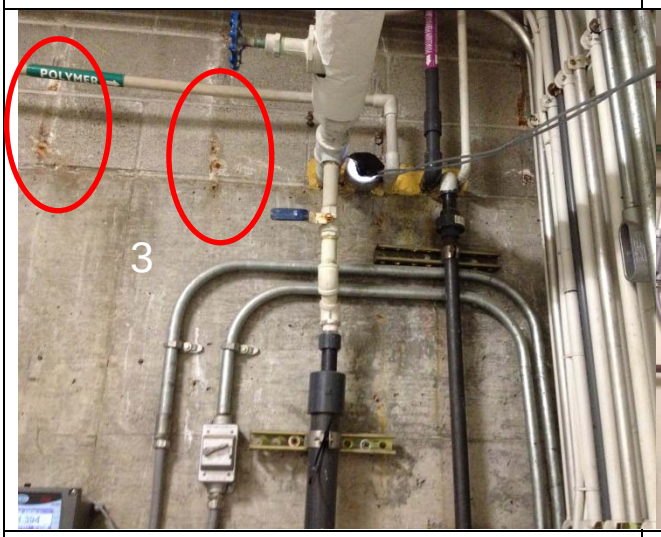
2. Limited access to panel.
3. Wall in break room gives limited access for maintenance activities.

Chemical Storage and Maintenance Room

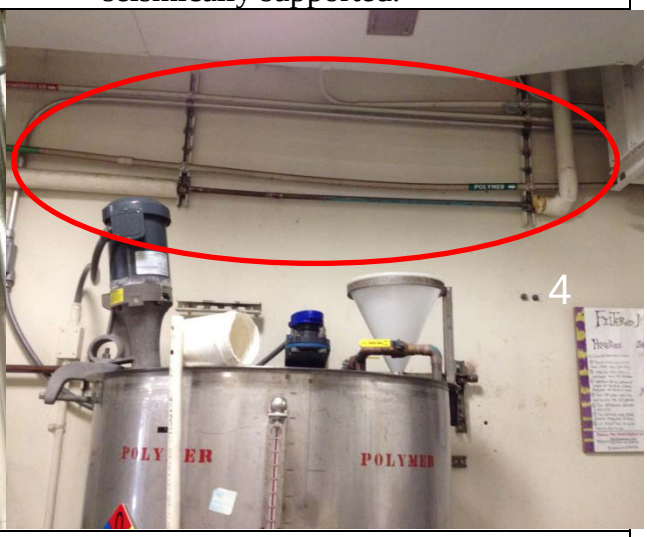


1. Concrete cracking observed on wall.

2. Drain may not be adequately sized to handle flows. Drain lines are not seismically supported.

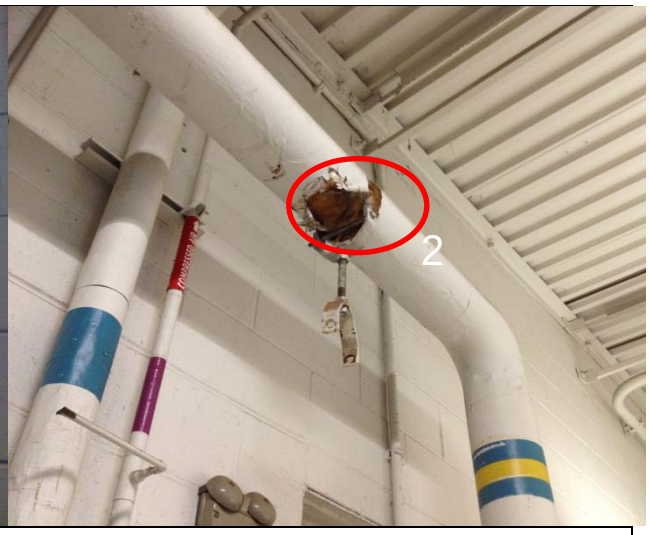


3. CMU deterioration.



4. Piping and conduits have insufficient support.

Chemical Storage and Maintenance Room Cont.



1. Paint failure.

2. Damage to pipe insulation; observed in multiple places.

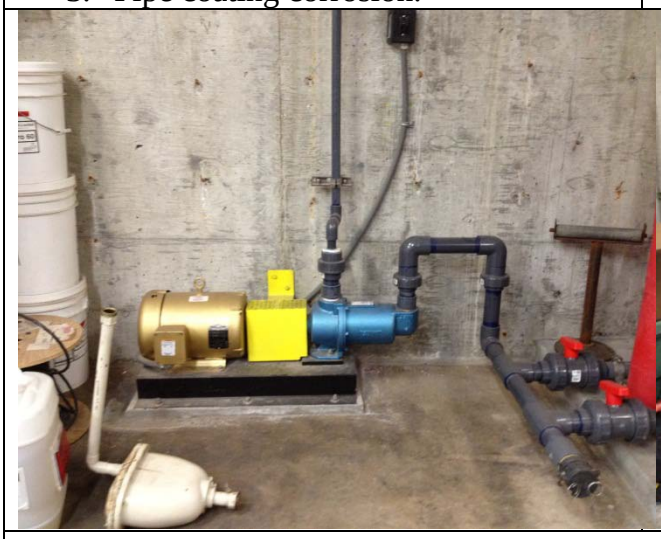
3. Paint deterioration.
4. CMU deterioration.
5. Pipe coating corrosion.

6. Door not sealed: HVAC problem.
7. Door frame rusting and deterioration.
8. Pipe coating corrosion.

Chemical Storage and Maintenance Room Cont.

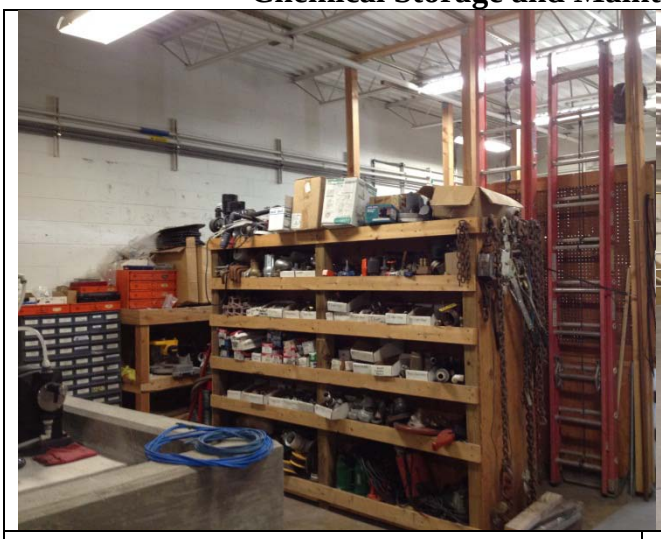
1. Paint deterioration.
2. CMU deterioration.
3. Pipe coating corrosion.

4. Pain failure close-up; found in multiple locations.

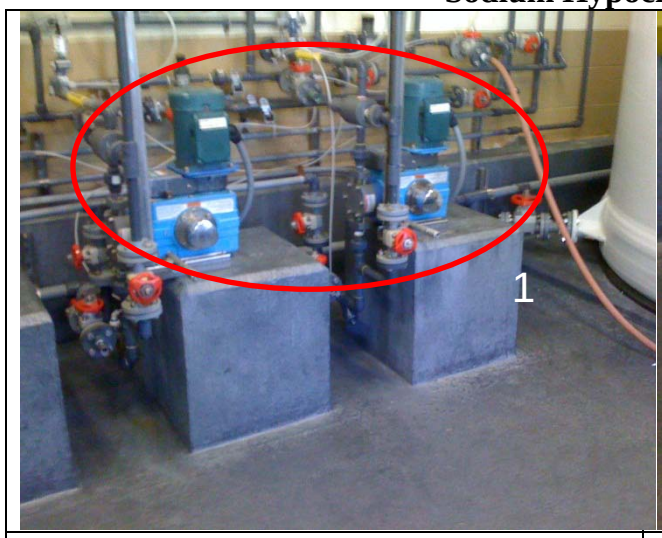
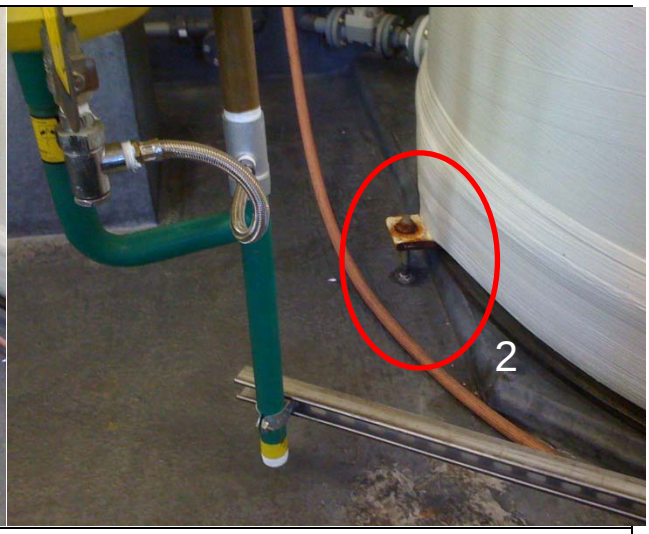


Inadequate storage and limited space.

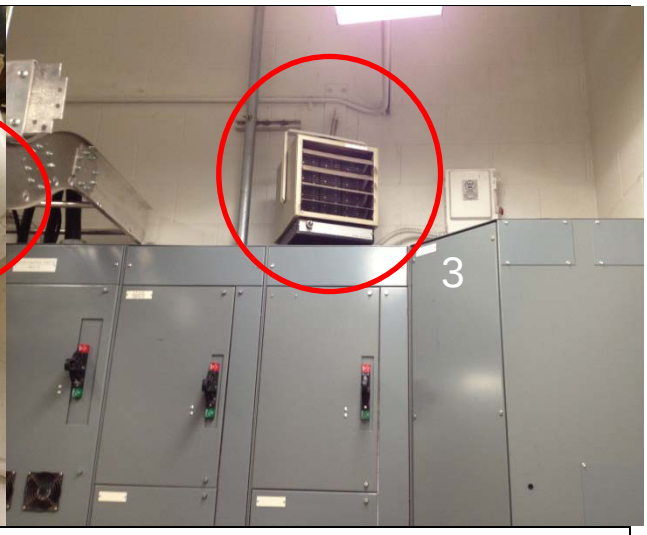
Chemical Storage and Maintenance Room (Continued)

	
Maintenance area co-located in the chemical room. Inadequate storage for equipment and parts.	Coagulant storage with containment area and metering pumps. Tanks cannot be removed or replaced without significant effort.
	
No spill containment for batching tanks.	1. Paint failure.

Sodium Hypochlorite Room

	
1. Sodium hypochlorite pumps.	2. Tanks may not be adequately anchored for seismic event. Corrosion observed on existing anchors.

Electrical Room



1. Paint failure.
2. Pipe wall penetration not seismically supported.

3. Limited HVAC control in electrical room.

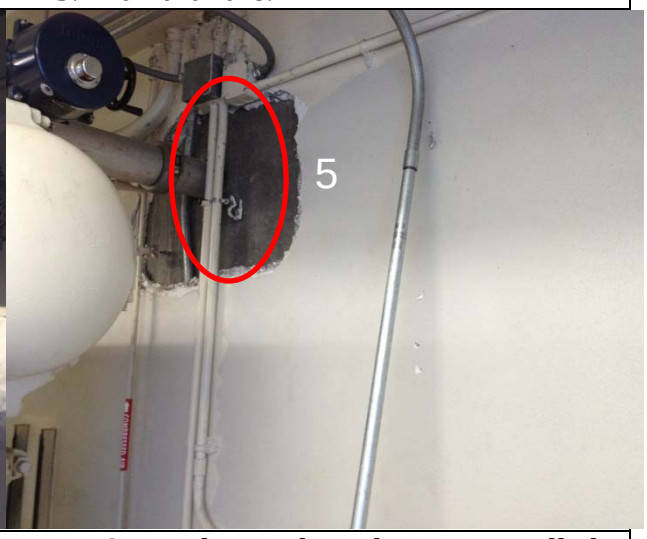
4. Pipe penetration wall insulation may need to be replaced.

5. Floor drain in electrical room located adjacent to filter gallery is undersized to handle emergency flows to prevent damage to electrical equipment.

Filter Galleries

1. Paint failure and corrosion underneath stairwell.

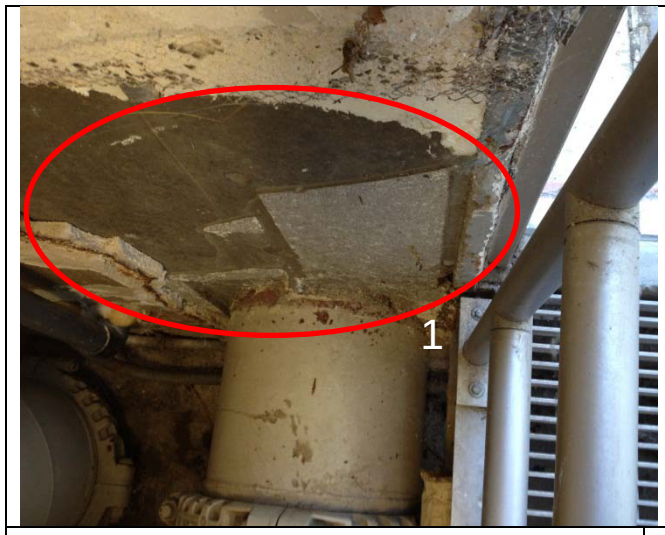
2. Flange coating failure.
3. Paint failure.



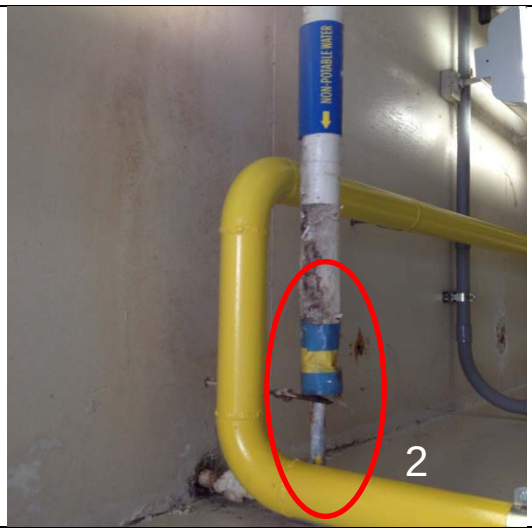
4. Pipe insulation deterioration and pipe hanger corroded.

5. Some electrical conduit was installed between wall cladding and wall. This presents a challenge to conduit maintenance, modification, and wall cladding removal.

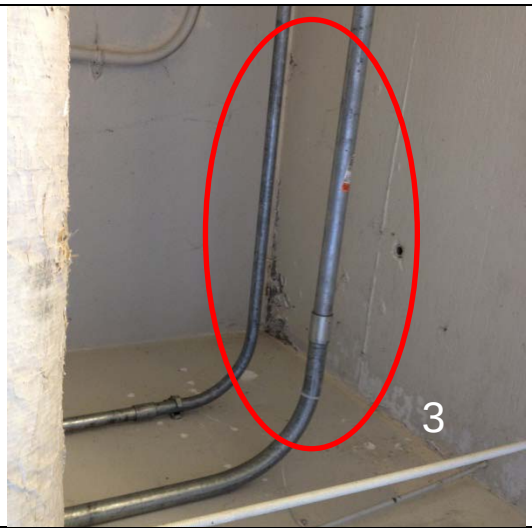
Filter Galleries Cont.



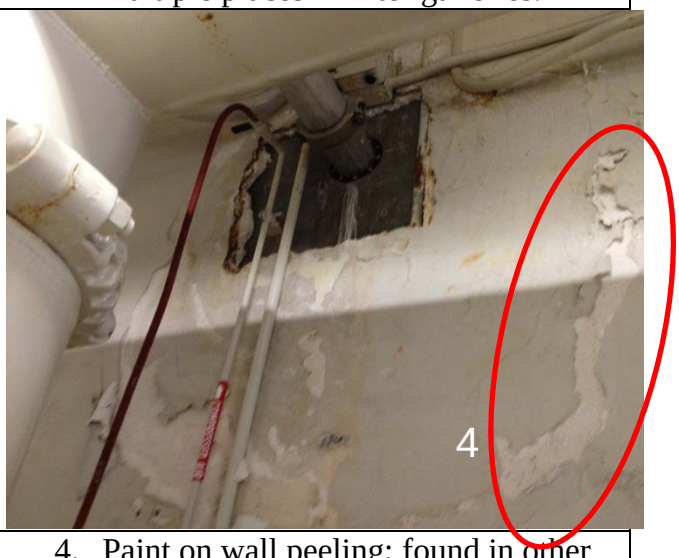
1. Cladding deterioration and exposed wire framework.



2. Damaged pipe insulation; found in multiple places in filter galleries.

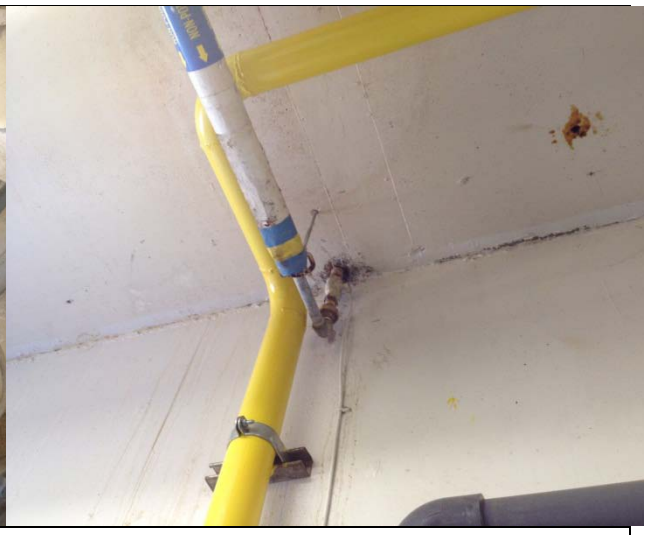
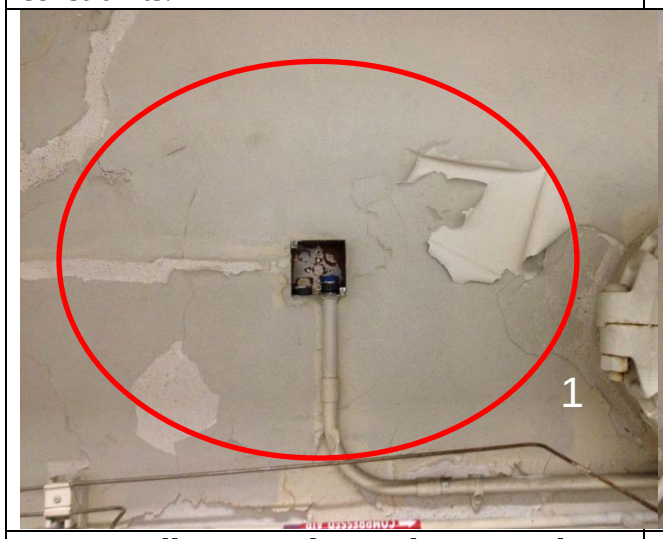


3. Unsupported conduits.



4. Paint on wall peeling; found in other locations within galleries.

Filter Galleries Cont.

	
Maintenance made challenging by space constraints.	Piping is not seismically supported.
	
1. Wall paint peeling and uncovered electrical junction box.	2. Drain line(s) may be undersized to handle major leaks or emergencies.

Filter Galleries Cont.

1. Pipe joint corrosion.

2. Pipe hanger showing signs of rust, pipe not insulated.

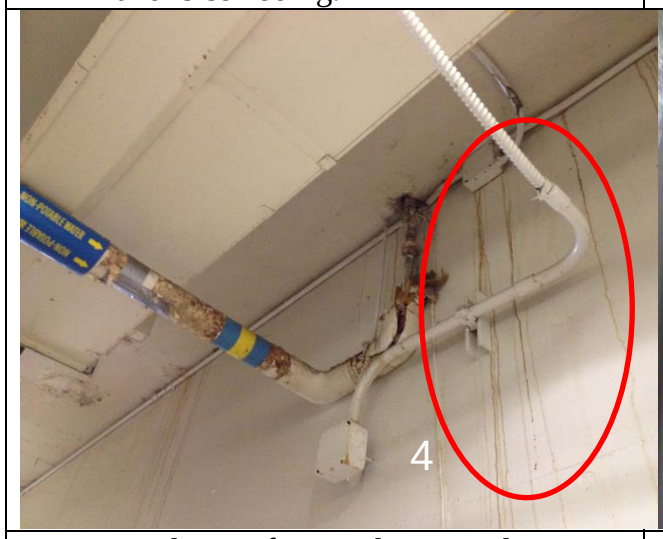
Challenging access for maintenance.

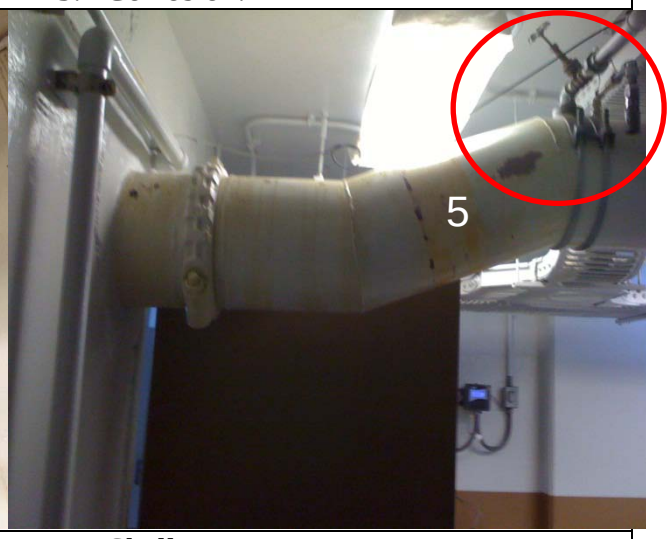
3. Exposed junction boxes.

Filter Galleries Cont.

1. Joint experiences intermittent leaking and is corroding.

2. Wall cladding and paint deterioration.
3. Corrosion.

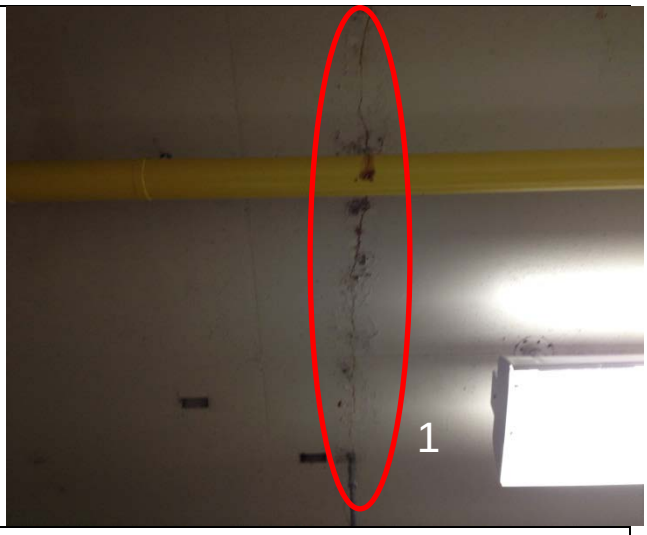
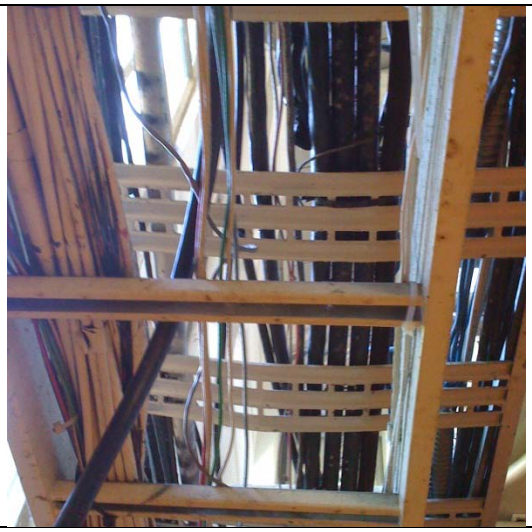
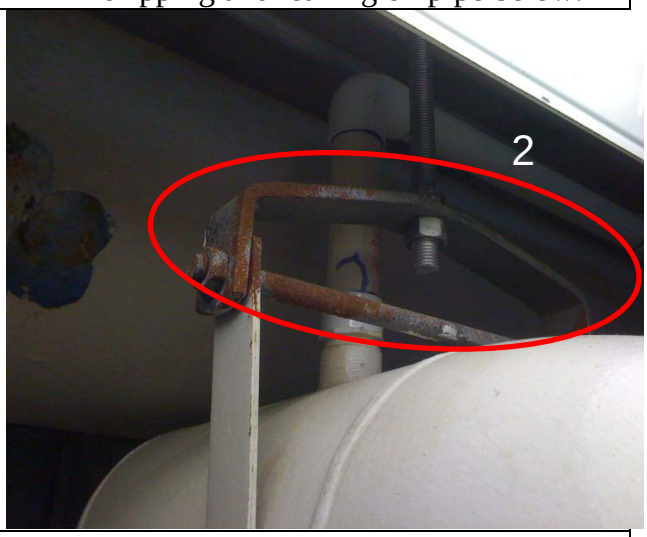




4. Evidence of water dripping along wall.

5. Challenging access to water tap.

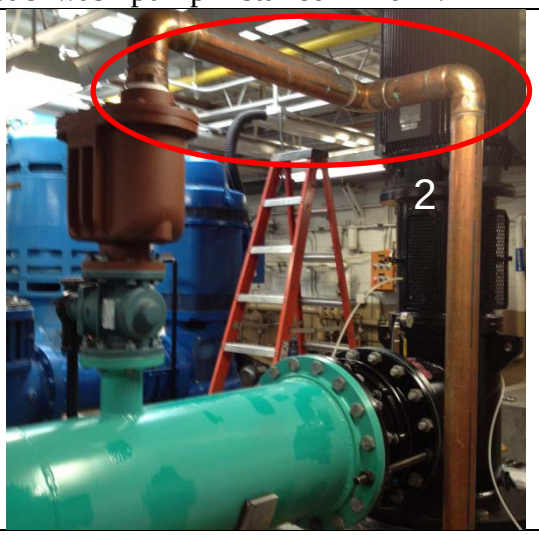
Filter Galleries Cont.

	
Pipe ceiling support not adequately designed for seismic event.	1. Crack in ceiling causing intermittent dripping and leaking on pipe below.
	
Cable tray not seismically supported.	2. Rusted pipe support.

High Service Pump Station Room

New backwash pump installed in 2012.

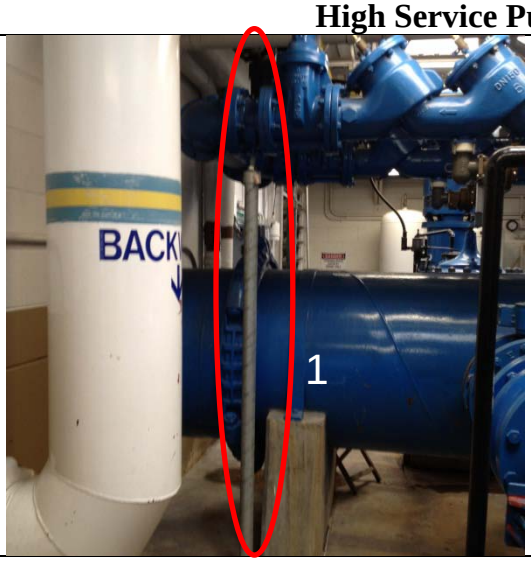
1. Inside wall core rusted.



2. Unsupported vent piping.

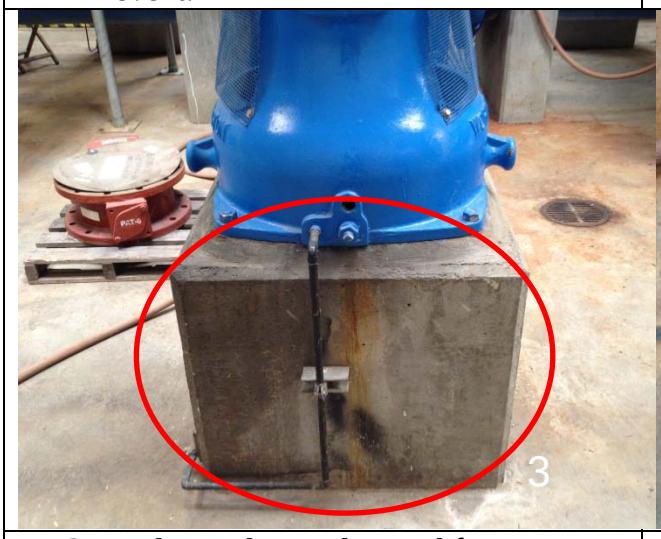
3. Temporary hose connection

High Service Pump Room Cont.

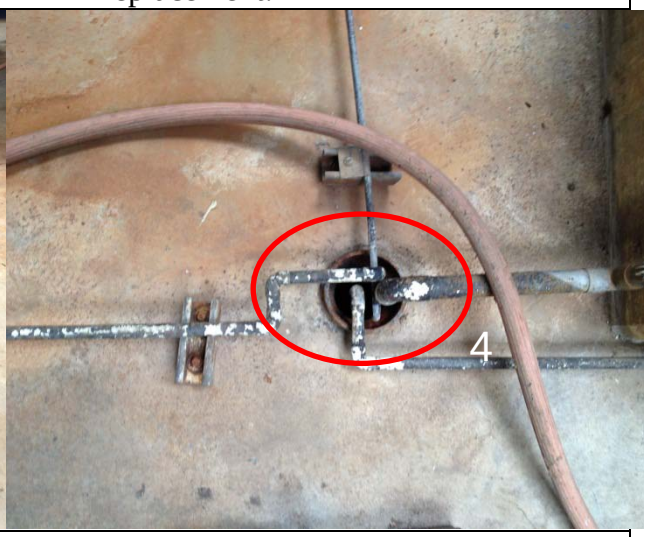


1. Pipe support not adequate for seismic event.

2. Label worn and in need of replacement.

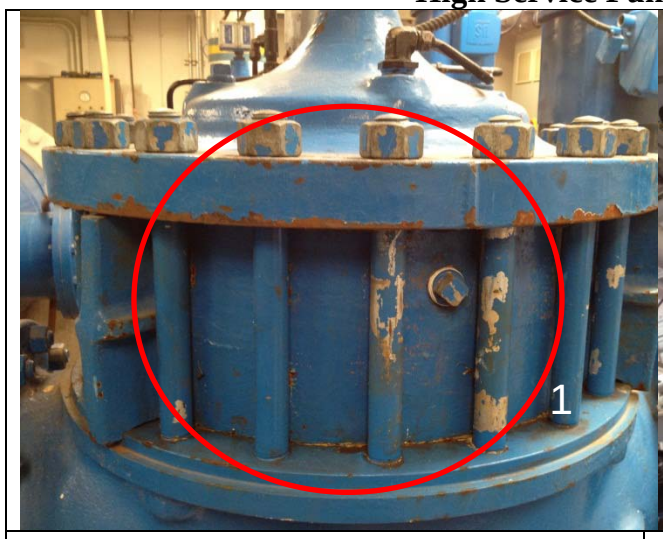
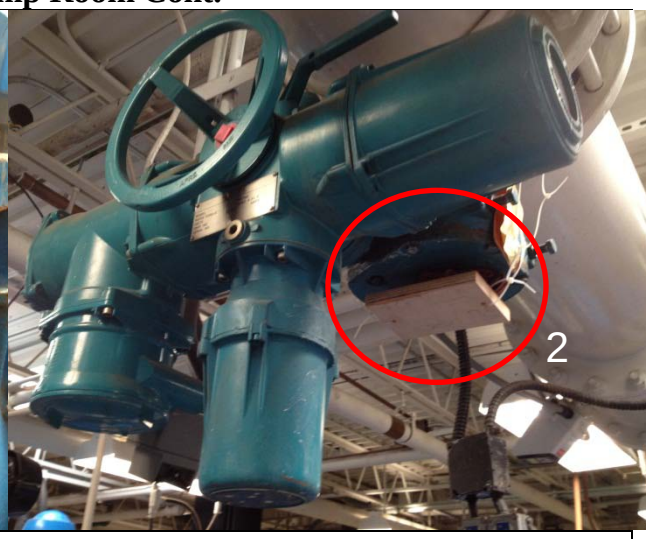


3. Pad not adequately sized for seismic event.

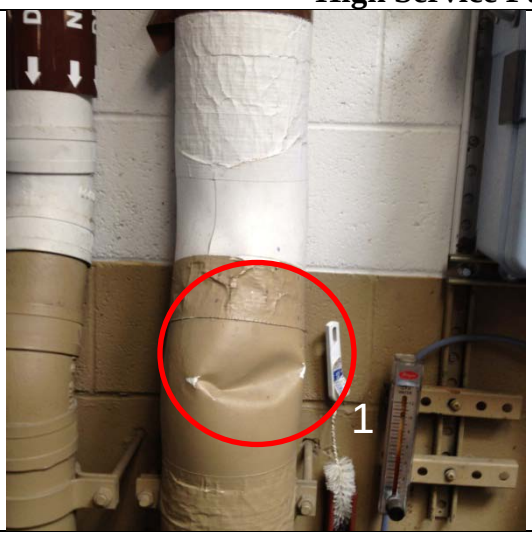


4. Drains in room undersized for emergency flows.

High Service Pump Room Cont.

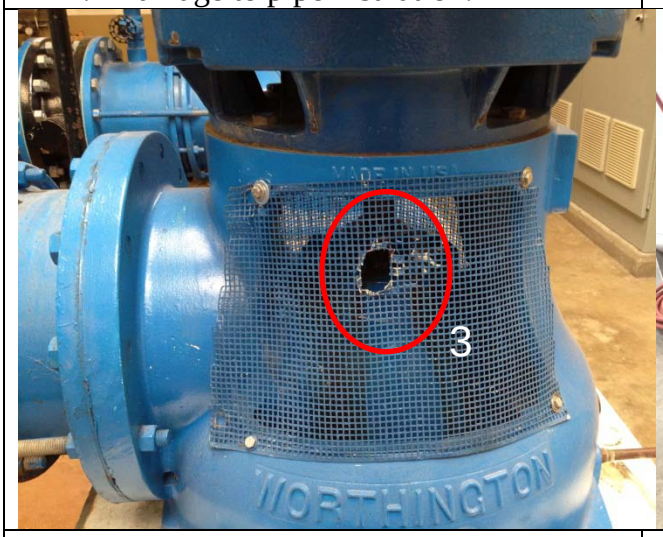
	
1. Corrosion.	2. Wooden block being held in place with plastic zip ties; needs permanent solution.
	
3. Valve may be failing. Difficult access and location for replacement and maintenance activities.	4. Temporary support may need to be replaced with something more permanent.

High Service Pump Room Cont.



1. Damage to pipe insulation.

2. Trip hazard identified.



3. Damage to screen poses a potential safety concern.

4. No hose bib or rack.

   As part of future work, the air compressors may be relocated to the chemical and maintenance area to alleviate risks associated with being located above the clearwell.

High Service Pump Room Cont.

1. Separate housing for battery back-up power supply to protect from potential leaks.

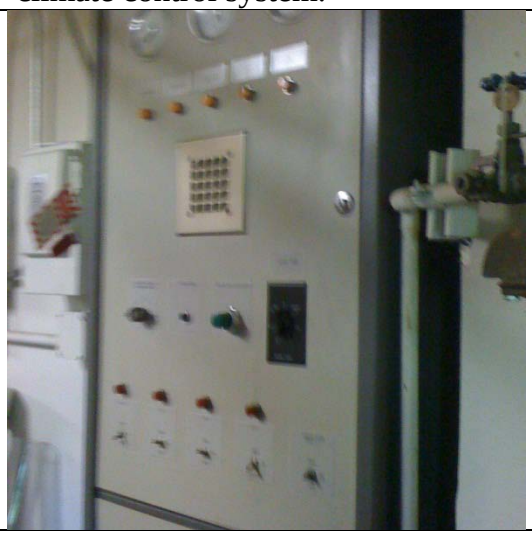
2. Poor access to wet well level

3. Tape and insulation may need to be replaced above doorframe.

HVAC

Main duct for air flow to operations building.

1. Example electric heater used in multiple locations around the plant. The plant lacks an optimized central climate control system.

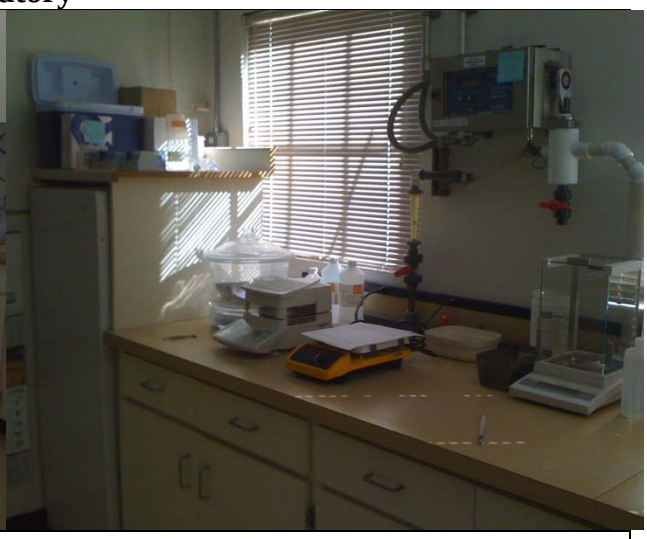


HVAC panels and components are dated and cannot be serviced or repaired easily by local contractors. It is difficult to find replacement parts.

River Intake

Existing raw water pump station. Additional space for pumps, but maintenance will be challenging in gallery below. The structure will need to be seismically upgraded as it is currently not laterally supported.

Laboratory



Limited bench-top area for staff to run samples or locate general lab equipment. Limited storage space for testing supplies.

Streaming current monitor.

Bench-top area being used for testing equipment; limited work area.

Laboratory Cont.

Finished water and raw water taps used for water quality monitoring and analysis.

Mill Pond

Geobags in use at Mill Pond.

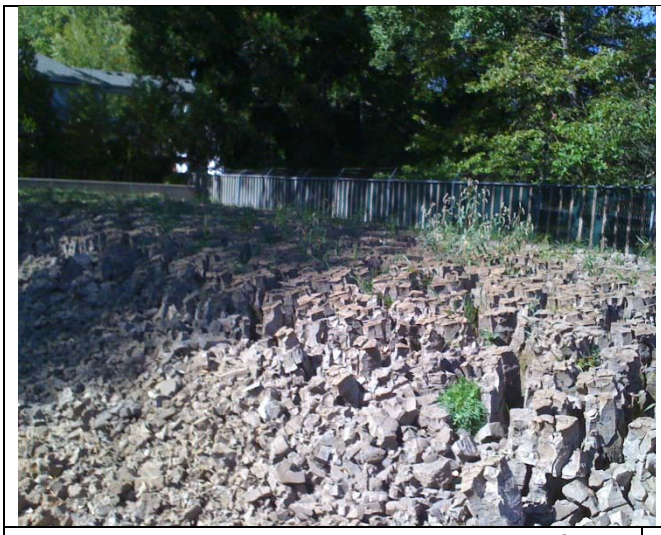
Dewatering polymer feed system.

Mill Pond overview.

1. Dredge used to collect solids for on-shore dewatering.

Effluent gate to Skunk Creek.

Solids Handling

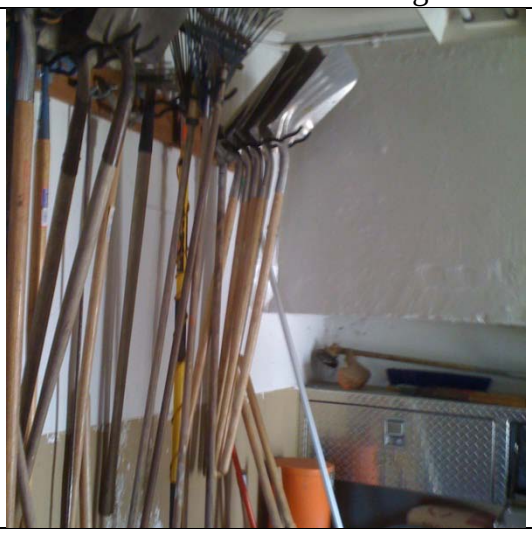
	
Solids being dried on-site at the plant before taken off-site for disposal.	Washwater equalization basin.
	
Cleaning and maintenance of washwater equalization basin is extremely labor-intensive because accessing the bottom is difficult. As a result, unwanted vegetation growth is common.	Space constraints on-site can lead to encroachment of access to valve boxes during solids dewatering.

Solids Handling Cont.

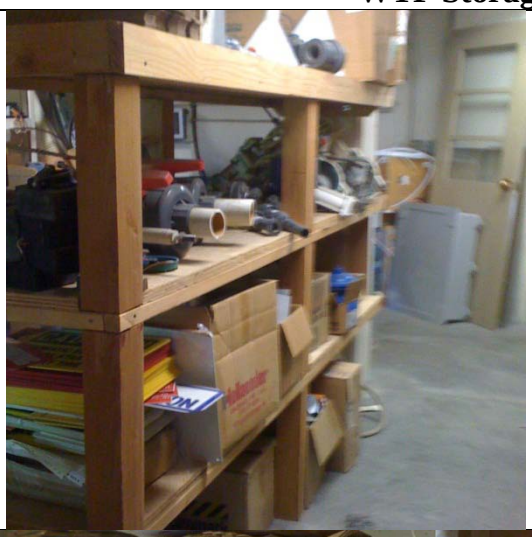
Washwater equalization basin pump(s) nearing end of useful life. Replacement will be challenging in compact space.

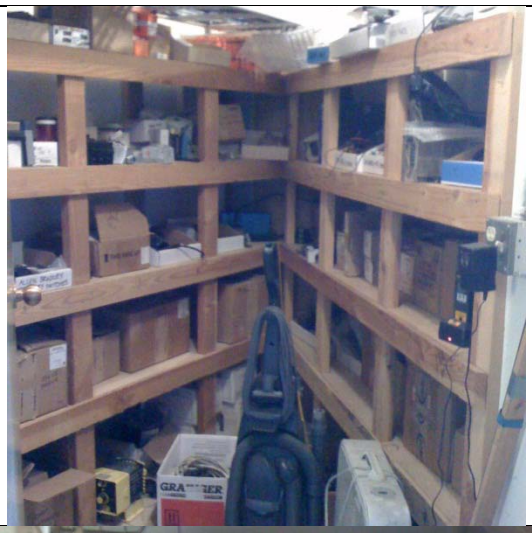
WTP Storage Areas

Storage space at the water treatment plant is limited. Crawl spaces and other non-traditional spaces have been utilized for storage of spare parts, maintenance supplies, equipment, etc. The plant also rents a storage locker off-site for additional items that cannot be stored locally.

	
Portion of server room used for storage.	1. Oil stored on-site.
	
Landscaping equipment housed on-site.	

WTP Storage Areas Cont.





Portions of the HVAC control attic used for storage.

Miscellaneous

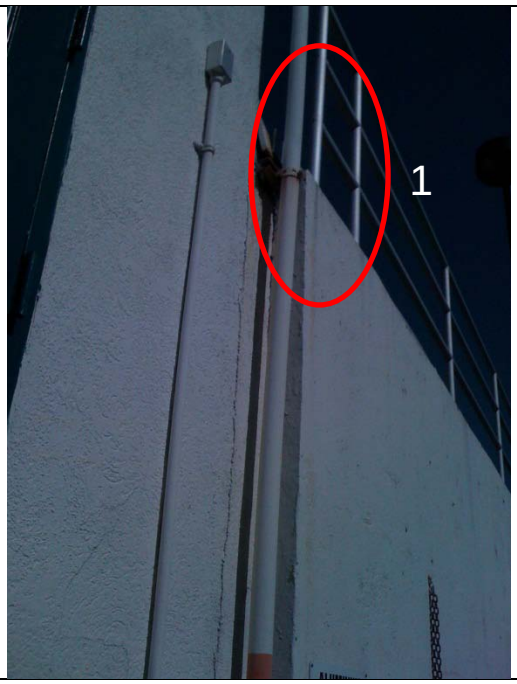
	
Portable generator not sized to run water treatment plant.	1. Exterior pipe supports corroding.

Table D-1
Inventory of Existing Grants Pass Water Treatment Plant Systems

Unit Process and Components	Quantity	Type	Manufacturer/Model	Capacity or Size
Screening				
Raw Water Intake Screen	4	Wedgewire Screen	Custom	4'-6" × 8'-0"
Wash System	1	Articulating Arm	Custom	25 – H3/8U-00120
Raw Water Pumping				
Raw Water Pumps				
Pump 1	1	Vertical Turbine	Worthington/ 15HH-340	75 HP/3,200 gpm/ 65 ft TDH
	1		AB Power Flex 700	
Pump 2	1	Vertical Turbine	Worthington/ 15HH-340	75 HP/3,200 gpm/ 65 ft TDH
Pump 3	1	Vertical Turbine	Worthington/ 15HH-340	75 HP/3,200 gpm/ 65 ft TDH
Pump 4	1	Vertical Turbine	Worthington/ 15HH-340	75 HP/3,200 gpm/ 65 ft TDH
	1		AB Power Flex 700	
Chemical Feed				
Coagulant				
Storage	2	Cylindrical Fiberglass		6,000 gal
Metering Pumps	3	PD Diaphragm	Grundfos/ Model DME 150	39.6 gph at 58 psi
	1	PD Diaphragm	Grundfos/ Model DME 60	15.9 gph at 145 psi
Air				
Compressor	2	Twin Units	Quincy/ Model 325	5 HP/ 19 cfm/ 130 gal receiver tank
After Drier 1	1		Hankison/ Model HPR50	50 scfm
Permanganate				
Storage		Stored in Metal Buckets		
Feed Unit	1	Hopper/Feeder/Mixer	BIF/ Model 25-06	$^{1}/_{3}$ HP/ 1,800 rpm

			AB Power Flex 700	
Polymer				
Storage	2	Stainless Steel Cylinder, Open-Top		290 gal
Mixing	1	Propeller	Neptune/ Model D-4.00	420 rpm
Mixing	1	Propeller	Philadelphia Mixer Co/ Model PG 34	420 rpm
Metering Pumps	1	PD Diaphragm	Grundfos/ Model DME 60	15.9 gpm at 145 psi
Hypochlorite				
Storage	3	FRP Cylinder	RTP, Inc.	2,300 gal
Pre-Chlorination Metering	1	PD Diaphragm	Wallace and Tiernan/ Encore 700	¾ HP/ 24 gph
Post-Chlorination Metering	1	PD Diaphragm	Wallace and Tiernan/ Encore 700	¾ HP/ 24 gph
Back-Up Metering	1	PD Diaphragm	Wallace and Tiernan/ Encore 700	¾ HP/ 24 gph
Transfer	1	Seal-less Magnetic	March/ Model TE-7.5K-MD	2 HP/ 3,435 rpm
Filtration				
Backwash Pump	1	Vertical Turbine with VFD	Peabody Floway/ Model 22-BLK	200 HP/ 7,000 gpm
	1		AB 1336 PLUS II	
	1	Vertical Turbine with VFD	Goulds Water Technology/ Model VIT-FFFM	150 HP/ 7,600 gpm/ 60 ft TDH
	1		AB Power Flex 700	
Surface Wash System				
Filters 1, 2, and 3	12	Stainless Steel	Leopold S Sweep	14.8 gpm per Sweep at 100 psi
Filters 4 and 5	8	Stainless Steel	Leopold S Sweep	14.8 gpm per Sweep at 100 psi
Filters 6, 7, and 8	12	Stainless Steel	Leopold S Sweep	14.8 gpm per Sweep at 100 psi

On-Line Monitoring				
Turbidity				
Raw Water	1	Digital – Integrated in SCADA	HACH Solitax Sc	0.001 to 4000 NTU
	1	Digital – Integrated in SCADA	HACH Surface Scatter 7	0.01 to 9999.9 NTU
Settled Water				
Sedimentation Basin 1	1	Digital – Integrated in SCADA	HACH 1720E	0 to 100 NTU
Sedimentation Basin 2	1	Digital – Integrated in SCADA	HACH 1720E	0 to 100 NTU
Sedimentation Basin 3	1	Digital – Integrated in SCADA	HACH 1720E	0 to 100 NTU
Filter Effluent				
Filter 1	1	Digital – Integrated in SCADA	HACH 1720E	0 to 100 NTU
Filter 2	1	Digital – Integrated in SCADA	HACH 1720E	0 to 100 NTU
Filter 3	1	Digital – Integrated in SCADA	HACH 1720E	0 to 100 NTU
Filter 4	1	Digital – Integrated in SCADA	HACH 1720E	0 to 100 NTU
Filter 5	1	Digital – Integrated in SCADA	HACH 1720E	0 to 100 NTU
Filter 6	1	Digital – Integrated in SCADA	HACH 1720E	0 to 100 NTU
Filter 7	1	Digital – Integrated in SCADA	HACH 1720E	0 to 100 NTU
Filter 8	1	Digital – Integrated in SCADA	HACH 1720E	0 to 100 NTU
Combined Filter Effluent	1	Digital – Integrated in SCADA	HACH 1720E	0 to 100 NTU
Plant Effluent	1	Digital – Integrated in SCADA	HACH 1720E	0 to 100 NTU
Chlorine Analyzers				
Mixed Water	1	Digital – Integrated in SCADA	HACH Cl-17	0 to 5 mg/L free Cl_2
Clearwell Influent	1	Digital – Integrated in SCADA	HACH CLF-10	0 to 10 mg/L free Cl_2
Clearwell Effluent	1	Digital – Integrated in SCADA	HACH Cl-17	0 to 5 mg/L free Cl_2
Flow Meters				
Raw Water	1	Venturi Differential Pressure	Barton/Fuji	0 to 125 in. water
Filter Effluent				
Filter 1	1	Orifice Differential Pressure	Barton	0 to 125 in. water
Filter 2	1	Orifice Differential Pressure	Bristol/ACCO Signature	0 to 100 in. water

Filter 3	1	Orifice Differential Pressure	Barton/Fuji	0 to 125 in. water
Filter 4	1	Orifice Differential Pressure	Barton	0 to 125 in. water
Filter 5	1	Orifice Differential Pressure	Barton	0 to 125 in. water
Filter 6	1	Orifice Differential Pressure	Barton	0 to 125 in. water
Filter 7	1	Orifice Differential Pressure	Barton/Fuji	0 to 125 in. water
Filter 8	1	Orifice Differential Pressure	Barton	0 to 125 in. water
Backwash	1	Electromagnetic Flow Meter	Danfoss Magflo Type MAG 5000	33 fps
Finished Water	1	Venturi Differential Pressure	Barton	0 to 125 in. water
Filter Head Loss				
Filter 1	1	Orifice Differential Pressure	Barton	0 to 125 in. water
Filter 2	1	Orifice Differential Pressure	Bristol/ACCO Signature	0 to 100 in. water
Filter 3	1	Orifice Differential Pressure	Bristol/ACCO Signature	0 to 100 in. water
Filter 4	1	Orifice Differential Pressure	Bristol/ACCO Signature	0 to 100 in. water
Filter 5	1	Orifice Differential Pressure	Bristol/ACCO Signature	0 to 100 in. water
Filter 6	1	Orifice Differential Pressure	Bristol/ACCO Signature	0 to 100 in. water
Filter 7	1	Orifice Differential Pressure	Bristol/ACCO Signature	0 to 100 in. water
Filter 8	1	Orifice Differential Pressure	Barton	0 to 125 in. water
pH				
Raw Water	1	pH Sensor	HACH pHD Sensor	2.0 to 14.0 pH
Clearwell	1	pH Sensor	HACH pHD Sensor	2.0 to 14.0 pH
Point of Entry	1	pH Sensor	HACH pHD Sensor	2.0 to 14.0 pH
High Service Pump Station				
Finished Water Pumps				
Pump 1	1	Vertical Turbine	Worthington/ Model 15HH-340	250 HP/ 3,500 gpm/ 210 ft TDH
Pump 2	1	Vertical Turbine	Fairbanks Morse/ Model 18HC	300 HP/ 4,000 gpm/ 210 ft TDH
Pump 3	1	Vertical Turbine	National Pump Company/ Worthington/ Model H14XHC	250 HP/ 3,500 gpm/ 220 ft TDH
	1		AB 1336 PLUS II	

Pump 3A	1	Vertical Turbine	National Pump Company/ Worthington/ Model H14XHC	250 HP/ 3,500 gpm/ 220 ft TDH
	1		AB 1336 PLUS II	
Pump 4	1	Vertical Turbine	Worthington/ Model 15HH-340	250 HP/ 3,500 gpm/ 210 ft TDH
Pump 5	1	Vertical Turbine	Worthington/ Model 15HH-277	250 HP/ 2,600 gpm/ 210 ft TDH
	1		AB 1336 PLUS II	
Waste Water				
Sewage Pumping				
Pumps	2	Submersible	EBARA/ Model 100DLMFU61.52	2 HP/ 1,800 rpm/ 80 gpm/ 30 ft TDH
Wastewater and Solids Equalization Tank	1	Concrete Tank	Custom	116,000 gal
Pump	1	Quick-disconnect Submersible	Peabody Barnes/ Model 6SEH2004	30 HP/ 1,500 gpm/ 36 ft TDH
Pump	1	Quick-disconnect Submersible	Peabody Barnes/ Model 6SE30034HL	60 HP/ 1,750 gpm/ 60 ft TDH
Pump	1	Quick-disconnect Submersible	Flygt/ Model CP3300.181-2200	60 HP/ 1,760 rpm
Plant Sump Pump	1	Quick-disconnect Submersible	Peabody Barnes/ Model 6SE-1004	12 HP/ 830 gpm/ 15 ft TDH
Solids Handling				
Storage	2	Polyurethane Cylinder, Open-Top	Snyder Industries, Inc/ Model HDPE	440 gal
Mixing	2	Propeller	Wingert/ Model WXL-20C/60	$^{1}/_{3}$ HP/ 1,725 rpm
Polymer Pump	1	Progressive Cavity	Moyno Inc/ Model 36701	2 HP/ 870 rpm
	1		AB Power Flex 700	
Solids Pump	1	Quick-disconnect Submersible	Flygt/ Model NP3153 HT 3153.181	15 HP/ 550 gpm/ 50 ft TDH
	1		AB Power Flex 700	

Turbidity	1	Digital – Integrated in Panel View	HACH Solitax Sc/ Model LXG 424.99	Solids 0.001 to 150 g/L
Flow Meter	1	Electromagnetic Flow Meter	Siemens/ Model Sitrans F M Magflo 5100, MAG 5000	33 fps
Level Sensor	2	Polymer Ultrasonic Level Sensor	Flowline/ Model Echospan LU81-510	8 in. to 16.4 ft
Level Sensor	1	Solids Ultrasonic Level Sensor	Siemens/ Model HydroRanger 200	1 ft to 50 ft
Dilution Valve	1	Ball Valve	Georg Fischer/ Model Type 546 with EA21 Actuator	2-inch dia PVC
Mixing Valve	2	Galvanized Steel Mixing Valve	Bellmer/ Model 36274-2	4-inch dia mixing chamber
Dredging				
Dredge	1	Log Pond Dredge	Liquid Waste Technologies, LLC/ Model Pit Hog Runt	480 V/ submersible pump 20 HP/ hydraulic motor 7.5 HP
	1		AB Power Flex 700	
Polymer Pump	1	Progressive Cavity	Moyno, Inc	1 HP/ 1-inch dia wet end
Sludge Pump	1	Slurry Pump on Dredge	Yeomans/ Model 9100X 4310L	20 HP/ 1,741 rpm/ 3 phase/ 460 V
Flow Meter	1	Electromagnetic Flow Meter	Toshiba/ Model LF434	0.3 to 10 mps velocity
Solids Monitor	1	Suspended Solids Monitor	Mobrey/ Model MSM400 Intelligent	0.5 to 50 percent solids
Mixing	1	Propeller	Neptune/ Model JG-6.1	1.5 HP/ 350 rpm
Storage	1	Polyurethane Cylinder, Open-Top	Poly Cal Plastics	500 gal
Mixing Valve	1	Galvanized Steel Mixing Valve	Bellmer/ Model 36274-2	4-inch dia mixing chamber

Table D-2
Observed Water Treatment Plant Deficiencies

Process, Building, or Area	Factors	Notes	Priority
Intake Structure	Seismic and Structural	Structure not laterally supported for seismic event. See Structural and Seismic Evaluation Report (2012) for additional information.	Medium
Intake Pumps	Process Hydraulic O/M	Firm capacity of 15 mgd with three pumps running. Operations and maintenance difficult due to space constraints. Additional pump installation to increase capacity will compound problems.	Medium
Mixing Basin	Seismic and Structural Hydraulic	Originally built in 1930's, does not meet current IBC requirements for "design earthquake." Cannot pass more than maximum plant flow without significant modifications.	Medium
Basin 1	Seismic and Structural Hydraulic Regulatory	Built in 1930's, does not meet current IBC requirements for "design earthquake." Visible cracks with leaking occurring from basin walls. Including other basins, cannot pass more than 21 mgd without significant modifications. Absence of solids removal system could impact disinfection and CT compliance.	High
Basin 2	Seismic and Structural Hydraulic Regulatory	Built in 1950's, does not meet current IBC requirements for "design Earthquake." Visible cracks with leaking occurring from basin walls. Including other basins, cannot pass more than 21 mgd without significant modifications. Absence of solids removal system could impact disinfection and CT compliance.	High
Basin 3	Seismic and Structural Hydraulic Process Regulatory	Built in 1980's, does not meet current IBC requirements for "design Earthquake." Including other basins, cannot pass more than 21 mgd without significant modifications. At high flows, basin short-circuits, reducing filter efficiency. Absence of sludge removal system could impact disinfection and CT compliance.	Medium
Filters 1, 2, 3, and Gallery	Seismic and Structural Process O/M	Built in 1930's, does not meet current IBC requirements for "design earthquake." Filters lack air scour resulting in increased maintenance and decreased plant efficiency. Cracks in walls and leaking observed.	Medium
Filters 4 and 5 and Gallery	Seismic and Structural Process O/M	Built in 1950's, does not meet current IBC requirements for "design earthquake." Filters lack air scour resulting in increased maintenance and decreased plant efficiency. Cracks in walls and leaking observed.	Medium

Filters 6, 7, 8, and Gallery	Seismic and Structural Process O/M	Built in 1980's, does not meet current IBC requirements for "design earthquake." Filters lack air scour resulting in increased maintenance and decreased plant efficiency. Cracks in walls and leaking observed.	Medium
Clearwell	Seismic and Structural Regulatory O/M	Does not meet current IBC requirements. Walls and supports have significant deterioration. See Structural and Seismic Evaluation Report (2012) for additional information. Limited volume has CT implications or disinfection. Poor confined space access.	High
Chemical and Maintenance Area	Seismic and Structural Environmental, Health, and Safety O/M	CMU blocks in load-bearing walls adjacent to basins are experiencing deterioration. Polymer system lacks containment. Due to lack of space, maintenance area is shared adjacent to chemical systems; very limited space for additional chemical storage. Ventilation and fire protection may not meet code requirements.	Medium
Sodium Hypochlorite Storage Room	Environmental, Health, and Safety O/M	Sodium Hypochlorite is very corrosive. Fittings need to be replaced with some frequency. Room lacks active ventilation. Only one of the three tanks can be removed from the building with ease. No room to add additional storage. Pumping systems do not have a failure alarm.	Low
Solids Handling	O/M Regulatory	Need updated NPDES permit for continued discharge to Skunk Creek. As system demands and solids production increase, current solids handling approach will need to be revised due to space restrictions and impact on plant efficiency.	Medium
Laboratory	O/M	Limited space to perform testing, and little to no space for additional testing equipment.	Low
Server Room	O/M	Server room does not have HVAC for climate control. Sensitive equipment subject to wide swings in temperature.	Low
Plumbing	Environmental, Health, and Safety O/M	Many plant drains are undersized to handle potential flows from surrounding equipment during maintenance or emergency events, and their condition, service, and discharge location are unknown.	Low
HVAC	O/M	System installed in 1980's and is in relatively good condition, but does not provide consistent heating, ventilation, and cooling throughout the plant. System has to be manually operated and adjusted. Control panel is antiquated and finding local service technicians is difficult.	Low
Electrical Systems	O/M Environmental, Health, and Safety	Most major electrical components are no older than 30 years. Some components have become harder to replace with age. Various cable trays and wiring are not seismically or structurally supported. Plant does not have a secure back-up power	Low

		supply, although City is in the process of obtaining a generator system.	
Major Process Piping	O/M	Sections of piping are beginning to show corrosion due to age. A lot of piping lacks structural and seismic pipe supports. Some pipes penetrate load-bearing walls.	Medium
Major Process Equipment	O/M	Various mechanical equipment will need to be replaced or rebuilt within the next 10 to 20 years. As the equipment continues to age, maintenance and repair cycles will shorten, causing increased labor costs and impacting plant efficiency.	Low

APPENDIX E

MSA
Murray, Smith & Associates, Inc.
Engineers/Planners
121 S.W. Salmon, Suite 900 ■ Portland, Oregon 97204-2919 ■ PHONE 503.225.9010 ■ FAX 503.225.9022

TECHNICAL MEMORANDUM

DATE: March 4, 2013

PROJECT: 12-1320.401

TO: Mr. Terry Haugen, Public Works Director
City of Grants Pass, Oregon

FROM: Brian Ginter, P.E.
Michael McKillip, P.E., Ph.D.
Murray, Smith & Associates, Inc.

RE: Long-Term Water Demand Projections

REGISTERED PROFESSIONAL ENGINEER 62947 OREGON JANUARY 11, 2005 BRIAN M. GINTER
RENEWS 6-30-13

REGISTERED PROFESSIONAL ENGINEER 65640 OREGON DECEMBER 29, 2009 MICHAEL L. McKILLIP
RENEWS 12-31-14

Introduction

The City of Grants Pass (City) authorized Murray, Smith & Associates, Inc (MSA) to prepare updated long-term water demand projections for the City's municipal drinking water supply. The purpose of this memorandum is to document the analysis, methodology and projections of water demand for both 20- and long-range planning horizons. The projections presented in this memorandum were developed considering the City's historical population, historical and present water demand characteristics, as well as other local and regional planning data. The water demand projections will be used as the basis for two water system planning projects: the Water Treatment Plant Facilities Plan Update, and an update of the City's Water Management and Conservation Plan. Long-range forecasting of demands will

also serve as the basis for the City's request for Extensions of Time to put water rights permits to beneficial use. It is anticipated that further refinement of these water demand projections needed to support the analysis of demands by pressure zone and the analysis of saturation development condition for the anticipated 2012 Urban Growth Boundary (UGB) expansion will be completed as part of the upcoming Water Distribution System Master Plan Update. These refined water demand projections are to be prepared under a separate memorandum.

Current and Future Service Area

The City currently provides water service to a population of approximately 34,756 people primarily within the existing City limits. The City limits encompass an area of approximately 7,000 acres, and include most of the area within the existing City UGB. The City began a process to expand the UGB in 2006 and it is anticipating that the completion of the process will occur by the end of 2012. The proposed expansion plan will be reviewed and jointly approved by the City and the Josephine County Board of Commissioners and subsequently approved by the State of Oregon Department of Land Conservation and Development. Based on a current analysis by the City's Community Development Department, it is anticipated that approximately 1,200 acres will be added to the UGB providing a 20-year land supply.

The City also provides water service to approximately 105 residential and commercial acres in the North Valley area located north of the City limits along Interstate Highway 5, and southeast of the unincorporated community of Merlin. Long-term future growth in this area is anticipated to be served by the City.

There are some adjacent developed areas outside the City limits that are not served by the City. These areas include the Rogue Community College to the west and unincorporated County areas to the southeast of the City. There is a potential for these areas to be served by the City in the future.

There are no significant jurisdictional constraints that would prevent long-term continued expansion of the UGB as the City grows. The City is not adjacent to any other municipality and while there are some mountainous areas not ideally suited for development, there are no significant topographic restrictions to the City's ultimate expansion.

Historical and Future Population Estimates

General

Estimates of the current and anticipated population within the water service area were developed through a review of existing City of Grants Pass planning data, previous water supply planning efforts, census data and Josephine County population forecasts. For planning purposes, the existing population within the City limits and the population of the water service area are assumed to be equal.

Historical and Existing Population

Historical City population data was obtained from the Portland State University's Population Research Center certified population estimates. The Population Research Center produces the annual population estimates for the State of Oregon and its counties and cities. These estimates, made July 1 of each year, are widely used for planning purposes. Table 1 summarizes historical and current populations within the City and for all of Josephine County. Figure 1 graphically illustrates this historical population data. From the year 2000 through 2010, the population in the City grew at an average annual rate of 4.0 percent. For the same period, Josephine County grew at a lower annual average rate of 0.85 percent. The City grew from 23,170 to 34,533 people and Josephine County (including the City) grew from 76,050 to 82,775 people. As the entire County added fewer people than the City added during the 10-year period, the overall demographic trend within Josephine County indicates a net shift in population into the City from the rural areas of Josephine County.

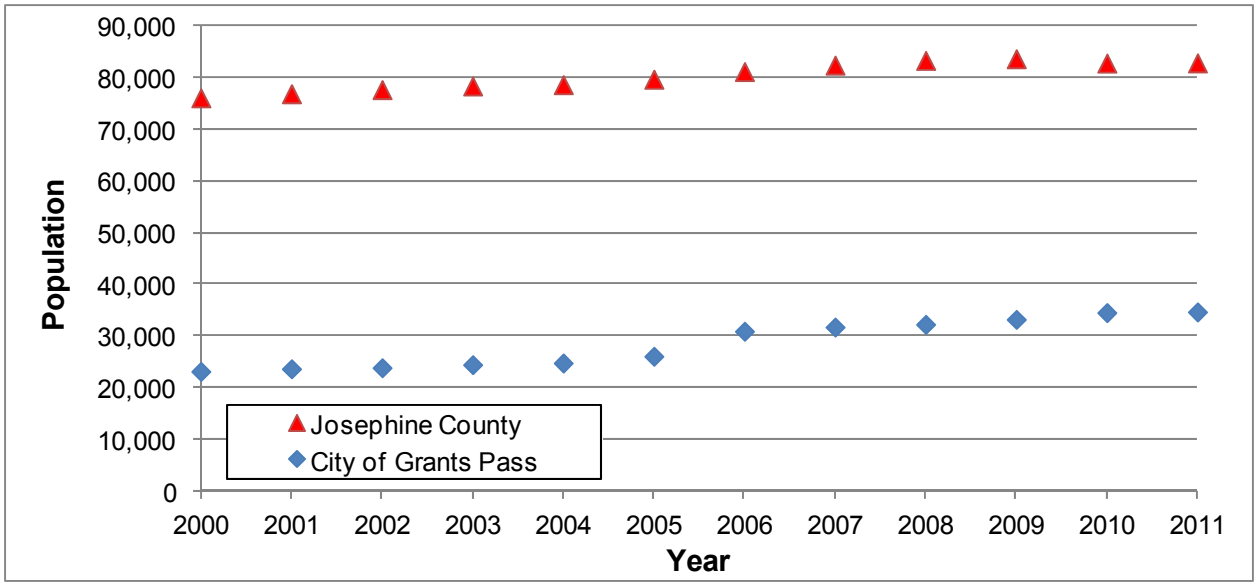
The City provided water to 36 residential properties in the North Valley area in 2011. At the City's residential density of 2.68 people per dwelling unit, the North Valley area is estimated to contribute a population of 96 to the City's service area.

Table 1
Historical City and County Population Summary

Year	City of Grants Pass		Josephine County		Percent of County Population
	Population	Percent Annual Growth	Population	Percent Annual Growth	
2000	23,170		76,050		30%
2001	23,670	2.2%	76,850	1.1%	31%
2002	23,870	0.8%	77,650	1.0%	31%
2003	24,470	2.5%	78,350	0.9%	31%
2004	24,790	1.3%	78,600	0.3%	32%
2005	26,085	5.2%	79,645	1.3%	33%
2006	30,930	19%	81,125	1.9%	38%
2007	31,740	2.6%	82,390	1.6%	39%
2008	32,260	1.6%	83,290	1.1%	39%
2009	33,225	3.0%	83,600	0.4%	40%
2010[1]	34,533	3.9%	82,775	-1.0%	42%
2011	34,660	0.4%	82,820	0.1%	42%

Note: 1. 2010 population estimates are adjusted to reflect 2010 Census data.

Figure 1
Historical City and County Population Summary



Population Forecast

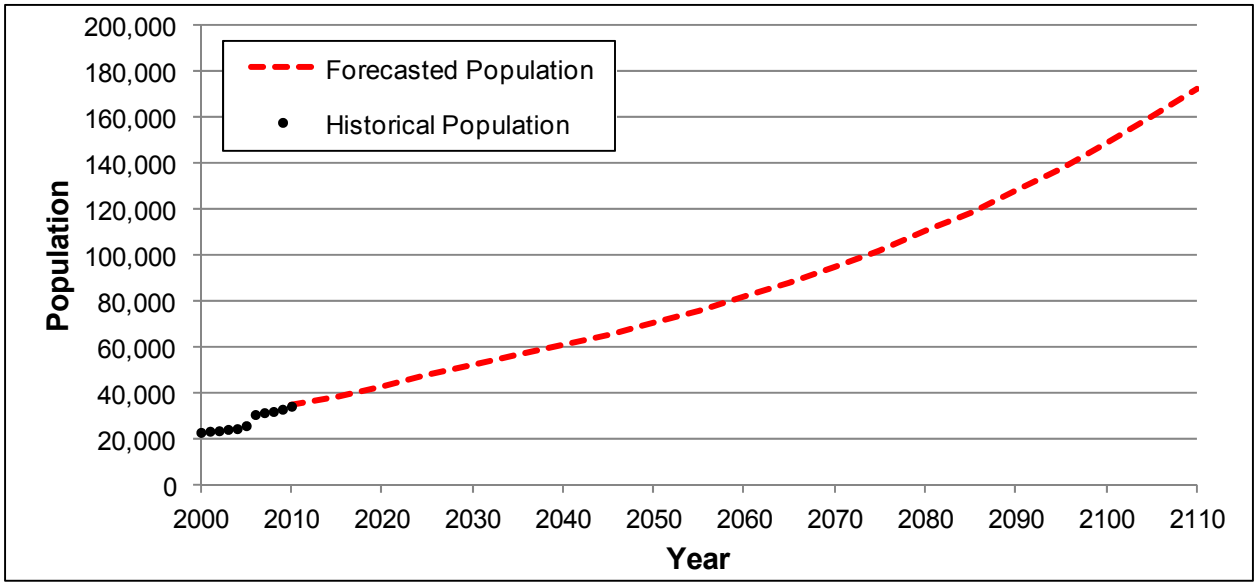
Planning studies have been prepared that forecast long-term population growth rates for Josephine County and the City. The Office of Economic Analysis reported a forecasted annual average growth rate for Josephine County from 2010 to 2040 of 1.1 percent. The relative historical growth rates of the City and Josephine County suggest that the 1.1 percent rate projected for the whole county is much lower than the recent and historical population growth rate of the City. The City's current Comprehensive Plan contains forecasted annual average population growth rates of 2.2 percent for the period 2007 through 2027 and 1.51 percent for the period 2007 through 2057. These rates are below the 2000 through 2010 actual growth rate of 4.0 percent.

Given the historically high recent growth rates of the City, a 2.2 percent annual growth rate is assumed through 2014. Thereafter, the annual growth rate is assumed to decline at a rate of 0.1 percent every five (5) years until it reaches an annual rate of 1.5 percent in 40 years. This results in a 20-year average annual growth rate of approximately 2.05 percent and a 50-year average annual growth rate of approximately 1.78 percent. The population projections using this approach are generally consistent with those in the City's Comprehensive Plan which projects a population of 54,540 in 2029 and 79,275 in 2057. The population forecast through a 100-year planning horizon to 2110 is reported in Table 2 and illustrated in Figure 2.

Table 2
Population Forecast Summary

Year	Service Area Population	Average Annual Growth Rate
2010 (Census Estimate plus North Valley Estimate)	34,649	2.2%
2015	38,632	2.1%
2020	42,862	2.0%
2025	47,323	1.9%
2030	51,993	1.8%
2035	56,844	1.7%
2040	61,843	1.6%
2050	72,125	1.5%
2060	83,704	1.5%
2070	97,142	1.5%
2080	112,738	1.5%
2090	130,837	1.5%
2100	151,841	1.5%
2110	176,218	1.5%

Figure 2
Historical and Forecasted City Population



The population forecasts presented in this memorandum are based on a review of historical population trends within the City and Josephine County and an extrapolation of population projections developed by the City for the purposes of land use and economic forecasting. The long-range forecasts presented do not consider potential future external influences on growth rates such as limitations on developable land, changing economic conditions, large shifts in demographic characteristics, and other factors. The projections provide an appropriate basis for long-term water system planning. It is expected that the accuracy of this forecast will decline significantly beyond a 20-year planning horizon as external influences not considered impact growth patterns.

Water Demand

General

Existing and future water demand estimates were developed following a review of historical water demand data provided by the City and population forecasts presented above. The term "water demand" refers to all the water requirements of the system including domestic, commercial, municipal, institutional as well as unaccounted-for water. A given water demand at any one time includes the sum of production from the City's Water Filtration Plant (WFP) plus the outflow from storage reservoirs. Demands are discussed in terms of gallons per unit time such as million gallons per day (mgd) or gallons per minute (gpm). Demands are also related to per capita use as gallons per capita per day (gpcd). Terminology used in this section to describe water usage characteristics are defined below:

Average Daily Demand (ADD): The Average Daily Demand is the total volume of water produced in a given year divided by 365 days. ADD is often used to forecast water volumes on an annual basis for estimating power costs, water revenue, and other considerations.

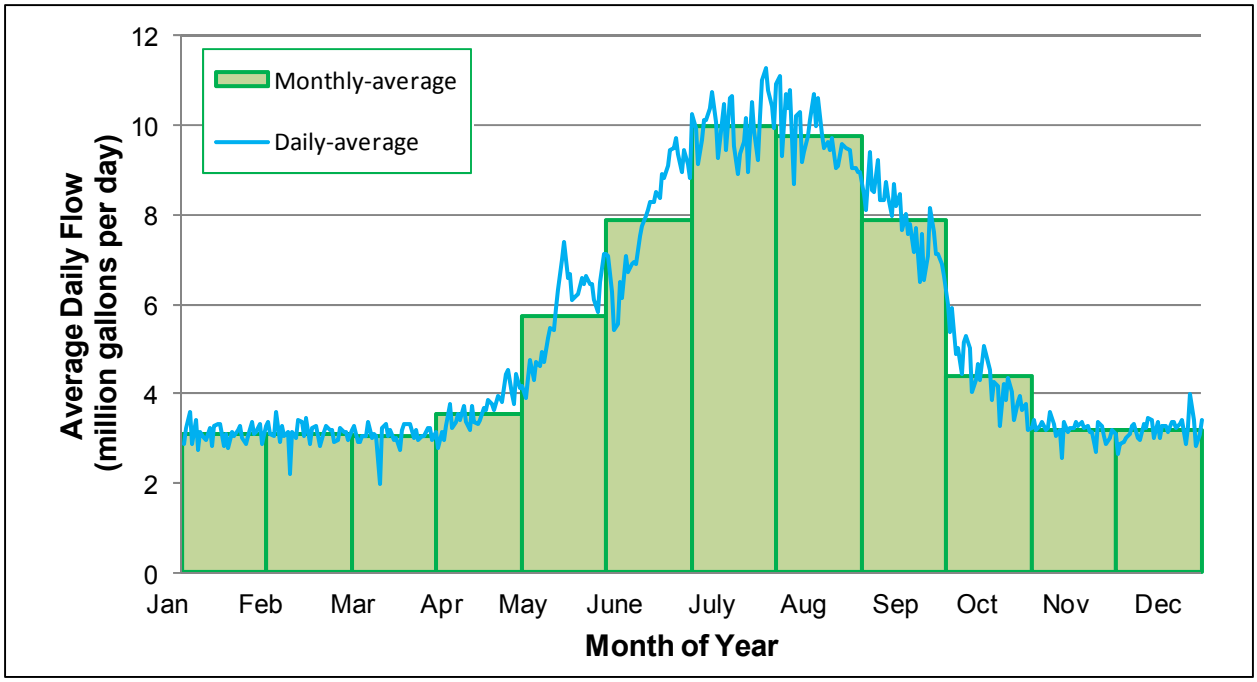
Peak Season Demand (PSD): Peak Season Demand is the average daily demand for the 122 days of the peak water use season; defined as June 1st to September 30th. The PSD reflects summer season outdoor water use patterns.

Maximum Day Demand (MDD): The Maximum Day Demand is the largest volume of water used, through production and changes in reservoir storage, in any single day of the calendar year. MDD is typically used to size the capacity of supply sources, treatment facilities, transmission piping, pumping facilities and finished water storage facilities. MDD usually occurs in the July to August months in the Pacific Northwest and is associated with increased outdoor water use on the hottest days of the year.

Peaking Factor: The ratio of the MDD to the ADD is commonly described as the peaking factor.

Figure 3 graphically presents both daily demand records and average monthly demands based on 5 years of production records. This figure illustrates the daily and seasonal variations in water demand for the water system.

Figure 3
Historical Average Demand



Historical Water Demand

The City records daily production at the WFP which are used to generate historical water demand statistics. Based on the historical average population presented above and water usage patterns, the water service area's average daily demand over the last five years has been between 5.0 and 5.8 mgd with an average day per capita consumption ranging between 145 gpcd and 184 gpcd. This is a typical range of average per capita daily demands for the region.

The historical MDD has been between 9.3 and 14.2 mgd with a maximum day per capita consumption ranging between 266 gpcd and 447 gpcd. The large range in maximum demand is due to the large number of variables that can influence summer season demand which include air temperature, precipitation, weekday versus weekend weather patterns, and other factors. Maximum day demands typically range from 250 to 450 gpcd using similar aggregate forecasting methods for similar sized communities in western Oregon and Washington. Table 3 summarizes historical water demand data for the years 2000 through 2011 by total production, per capita rates and peaking factor. The water demand characteristics for 2011 are anomalous in that the MDD is much smaller than the previous years. Figures 4 and 5 illustrate the historical water demand characteristic as both daily and per capita demands.

Figure 4
Historical Daily Water Demand Characteristics

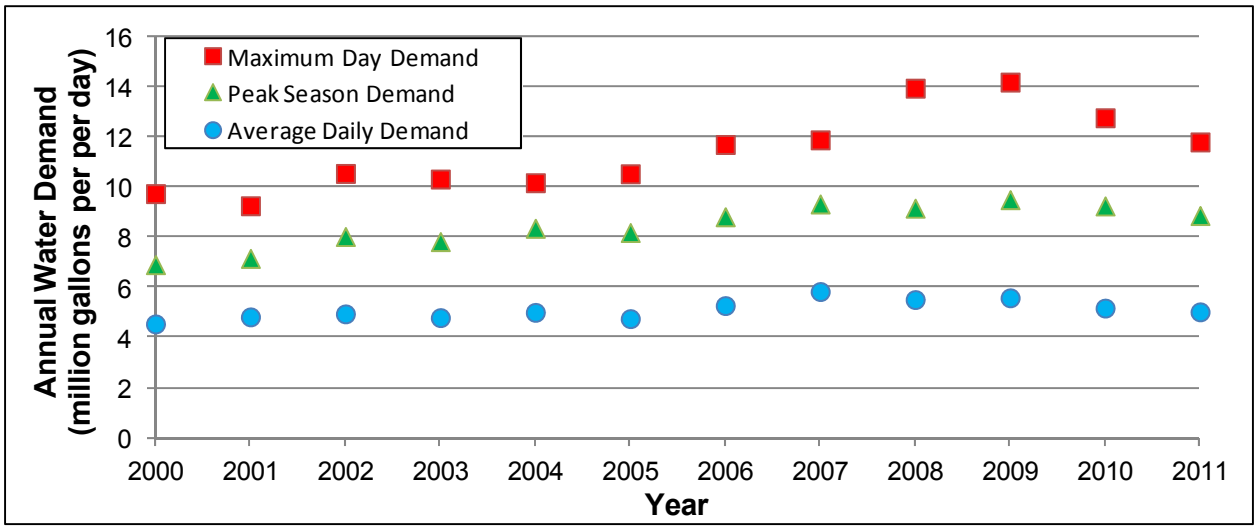


Figure 5
Historical Per Capita Water Demand Characteristics

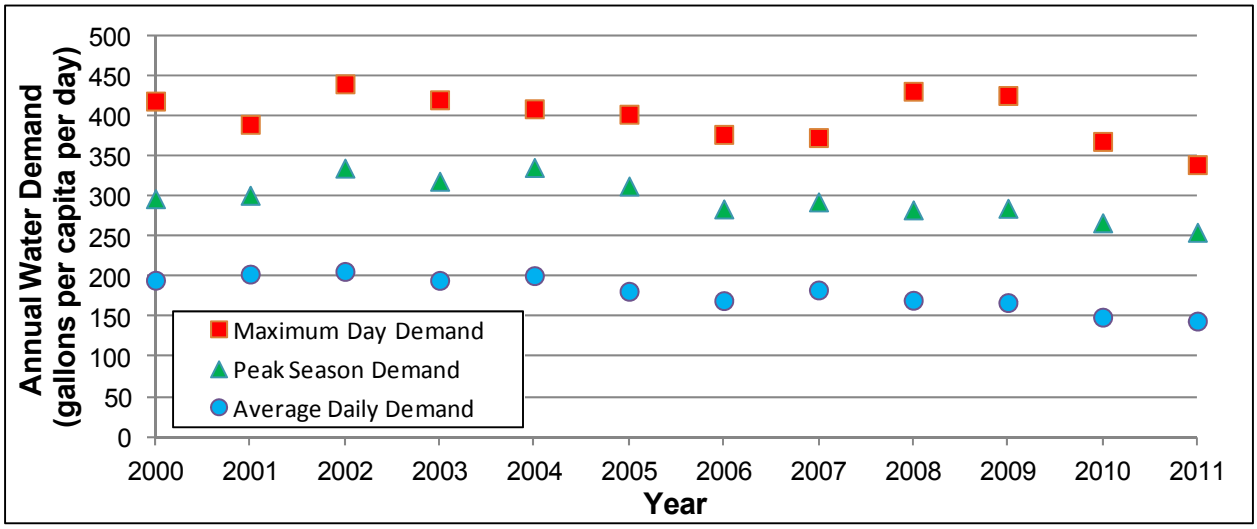


Table 3
Historical Water Demand Summary

Year	Water Service Area Population	Water Demand (million gallons per day)			Per Capita Water Demand (gallons per capita per day)			Peaking Factor	
		ADD	PSD	MDD	ADD	PSD	MDD	PSD	MDD
2000	23,249	4.5	6.9	9.7	195	296	419	1.52	2.14
2001	23,750	4.8	7.1	9.2	203	300	389	1.48	1.92
2002	23,951	4.9	7.9	10.5	206	328	440	1.59	2.13
2003	24,552	4.8	7.9	10.3	195	322	420	1.65	2.15
2004	24,873	5.0	8.1	10.2	201	327	409	1.63	2.03
2005	26,169	4.8	7.6	10.5	182	291	402	1.60	2.22
2006	31,015	5.3	8.4	11.7	170	272	377	1.60	2.22
2007	31,826	5.8	9.4	11.9	183	296	373	1.61	2.04
2008	32,346	5.5	9.0	13.9	170	277	431	1.63	2.53
2009	33,318	5.6	9.1	14.2	167	273	425	1.63	2.54
2010	34,632	5.2	8.6	12.8	149	248	368	1.66	2.47
2011	34,756	5.0	8.3	11.8	144	240	339	1.66	2.35
5-year average ('06-'10)		*5.5*	*8.9*	*12.9*	*168*	*273*	*395*	*1.63*	*2.36*
10-year average ('00-'10)		*5.1*	*8.2*	*11.4*	*184*	*294*	*405*	*1.60*	*2.22*

Notes:
1. Abbreviations: Average Daily Demand (ADD); Peak Season Demand (PSD); Maximum Daily Demand (MDD).
2. The water demand characteristics for 2011 are anomalous, and are not used to calculate historical averages.
3. Water service area population includes the North Valley area component.

Projected Water Demands

Projections of future water demands are determined based upon present and historical per capita water use characteristics and forecasted future population.

Water demand forecasts are used to ensure adequate supply and transmission capacity under a maximum day demand scenario. Major water infrastructure projects often take 5 to 10 years to complete as the City proceeds from identification of a deficiency through project planning, funding, design, bidding, award and construction. Based on a review of historical and current water use characteristics within the City's water service area, observation of regional and national water use trends, and anticipated future advances in water saving technology, the following water demand projection criteria are used:

<u>20-Year Planning Horizon (~2035)</u>

- Per capita average day demand assumed to be at the average rate over the 5 years from 2006 through 2010, 170 gpcd.
- Per capita maximum day demand assumed to be approximately 400 gpcd based on the average peaking factor of 2.35 over the 2006 through 2010 period.

<u>Beyond the 20-Year Planning Horizon</u>

- Beyond the 20-year planning period, it is more probable that water demand growth will not increase at the same rate as assumed for the near term planning purposes.
- The per capita water demand rate was assumed to decrease from 170 gpcd by 5 gpcd after each 5-year block such that a demand of 140 gpcd is achieved by 2065.

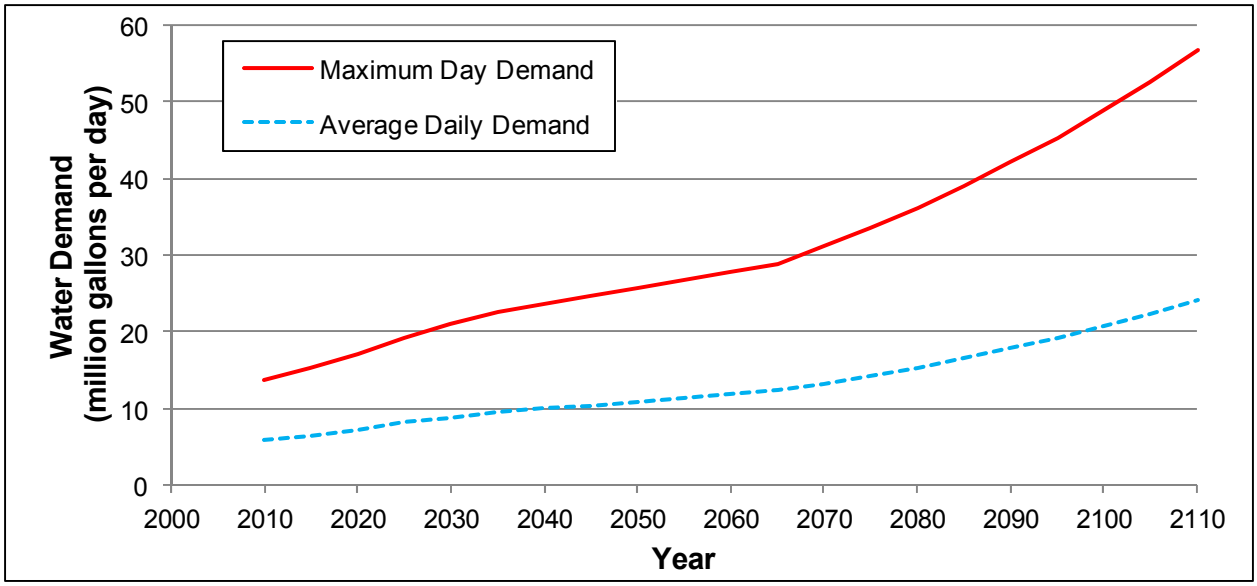
Table 4 presents a summary of population and water demand forecasts in five year increments through 2040 and in 10-year increments to the year 2110. The purpose of the forecasts presented in this memorandum is to provide a basis for planning of water supply and treatment needs. It is recommended that these projections be updated every 5 to 10 years to reflect current conditions and to support updates of capital infrastructure prioritization, funding and implementation.

Table 4
Population and Water Demand Forecasts Summary

Year	Service Area Population	AAGR (percent)	Per Capita Demand (gpcd)	ADD (mgd)	MDD (mgd)
2015	38,632	2.1%	170	6.6	15.5
2020	42,862	2.0%	170	7.3	17.1
2025	47,323	1.9%	170	8.0	18.9
2030	51,993	1.8%	170	8.8	20.8
2035	56,844	1.7%	170	9.7	22.7
2040	61,843	1.6%	165	10.2	24.0
2045	66,951	1.5%	160	10.7	25.2
2050	72,125	1.5%	155	11.2	26.3
2055	77,700	1.5%	150	11.7	27.4
2060	83,704	1.5%	145	12.1	28.5
2065	90,173	1.5%	140	12.6	29.7
2070	97,142	1.5%	140	13.6	32.0
2075	104,650	1.5%	140	14.7	34.4
2080	112,738	1.5%	140	15.8	37.1
2085	121,451	1.5%	140	17.0	40.0
2090	130,837	1.5%	140	18.3	43.0
2095	140,948	1.5%	140	19.7	46.4
2100	151,841	1.5%	140	21.3	50.0
2105	163,576	1.5%	140	22.9	53.8
2110	176,218	1.5%	140	24.7	58.0

Note: 1. Abbreviations: Average Annual Population Growth Rate (AAGR); Average Daily Demand (ADD); Maximum Daily Demand (MDD); million gallons per day (mgd); gallons per capita per day (gpcd)

Figure 6
Projected Water Demand



Summary

This memorandum presents historical and forecasted population and water demands. The current service area population is approximately 34,756 and the planning level ADD of 5.9 mgd and MDD of 13.9 mgd. By 2030, the population is forecasted to be approximately 51,993 and the projected ADD is 8.8 and the MDD is 20.8 mgd. The City of Grants Pass is anticipated to continue to expand and grow well beyond the UGB expansion currently being adopted. By 2110, the population is forecasted to be approximately 176,218 and the projected ADD is 25 and the MDD is 58 mgd. These projections are generally consistent with the Josephine County and City's current Comprehensive Plan projections. It is recommended that these projections will be updated every five (5) to 10 years to reflect current conditions and to support updates of capital infrastructure prioritization, funding and implementation.

BMG:mlm

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